



THURBER ENGINEERING LTD.

**FOUNDATION INVESTIGATION AND DESIGN REPORT
REPLACEMENT OF STRUCTURAL CULVERT No. 29-232/C
MUSKRAT CREEK CROSSING OF HIGHWAY 17
RENFREW COUNTY
W.P. 4113-01-01
AGREEMENT NUMBER: 4014-E-0014**

GEOCRES NUMBER: 31F-201

SUBMITTED TO

WSP CANADA

LOCATION:

LATITUDE: 45.58687°

LONGITUDE: -76.83145°

APRIL 2018

19-5161-263



Table of Contents

PART 1: FACTUAL INFORMATION

1	INTRODUCTION	1
2	SITE DESCRIPTION	1
3	SITE INVESTIGATION	2
3.1	Field Investigation	2
3.2	LABORATORY TESTING	3
4	DESCRIPTION OF SUBSURFACE CONDITIONS	4
4.1	Overview / General	4
4.2	Asphalt.....	4
4.3	Topsoil.....	4
4.4	Fill.....	4
4.5	Sand with Silt / Silty Sand	5
4.6	Clay	5
4.7	Silt with Sand	5
4.8	Silty Sand with Gravel Till	6
4.9	Granite Bedrock.....	6
4.10	Groundwater Conditions	6
5	MISCELLANEOUS	7

PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

6	GENERAL.....	8
6.1	Proposed Structure	8
6.2	Applicable Codes and Design Considerations.....	9
7	SEISMIC CONSIDERATIONS	9
7.1	Spectral and Peak Acceleration Hazard Values	9
7.2	Seismic Liquefaction Assessment.....	9
7.3	CHBDC Seismic Site Classification.....	10
8	GEOTECHNICAL ASSESSMENT.....	11
8.1	Design Considerations	11
8.2	Foundation Design Alternatives – Liquefaction Mitigation	11



8.3	Culvert Type Alternatives	12
8.3.1	Circular Pipes	12
8.3.2	Open Bottom Culvert	12
8.3.3	Closed Bottom (Box) Culvert.....	12
8.3.4	Pre-cast vs Cast-in-Place Concrete Culverts	12
8.4	Recommended Approach for Culvert Replacement	13
9	FOUNDATION DESIGN RECOMMENDATIONS	13
9.1	Culvert Foundation Bearing Resistances	13
9.2	Subgrade Preparation, Culvert Bedding and Backfilling	14
9.3	Lateral Earth Pressures	15
9.3.1	Static Lateral Earth Pressure Coefficients.....	15
9.3.2	Seismic Lateral Earth Pressure Parameters.....	16
9.4	Embankment Design and Reinstatement	16
9.5	Cement Type and Corrosion Potential	16
10	CONSTRUCTION CONSIDERATIONS	17
10.1	Excavations	17
10.2	Temporary Protection Systems.....	17
10.3	Dewatering	18
10.4	Erosion Protection.....	20
10.5	Construction Concerns	20
11	CLOSURE	21



APPENDICES

Appendix A	Borehole Locations and Soil Strata Drawings
	Preliminary General Arrangement Drawing
Appendix B	Record of Borehole Sheets
Appendix C	Laboratory Test Results
Appendix D	Seismic Cone Penetration Testing Results
Appendix E	Site Photographs
Appendix F	Comparison of Culvert Replacement Alternatives
	Comparison of Culvert Design Alternatives
Appendix G	GSC Seismic Hazard Calculation
	Figure 1: Comparison of The CPT Test Results
	Figure 2: Comparison of CRR to CSR
	Figures 3 & 4: Slope Stability Analysis Results
Appendix H	List of Referenced Specifications

**FOUNDATION INVESTIGATION AND DESIGN REPORT
REPLACEMENT OF STRUCTURAL CULVERT No. 29-232/C
MUSKRAT CREEK CROSSING OF HIGHWAY 17
RENFREW COUNTY
W.P. 4113-01-01
AGREEMENT NUMBER: 4014-E-0014**

GEOCRES NUMBER: 31F-201

PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual data obtained from a foundation investigation conducted by Thurber Engineering Ltd. (Thurber) for the replacement of the Muskrat Creek Culvert located on Highway 17, within Renfrew County. Thurber carried out the investigation as a subconsultant to WSP Canada (WSP) as part of Agreement No. 4014-E-0014.

No previous foundation investigation information for the subject culvert was available. A General Arrangement (GA) drawing and base plan mapping were provided by WSP for the preparation of this report.

The purpose of this investigation was to explore the subsurface conditions at the site and, based on this data, provide a borehole location plan, record of boreholes, a stratigraphic profile, laboratory test results and a written description of the subsurface conditions.

2 SITE DESCRIPTION

Culvert 29-232/C is located on Highway 17, approximately 5.7 km east of Cobden, Ontario. The location of the culvert is shown on the inset Key Plan on Drawing No. 1 in Appendix A.

The existing culvert is a cast-in-place, concrete, open bottom, rigid frame culvert, with an internal span of 3.1 m, a height of 1.8 m and an approximate length of 25 m. Water flow is from north to south below the highway. The December 2017 Preliminary GA drawing, also provided in Appendix A, indicates that the elevation of the top of the existing stream bed ranges from Elevation 146.7 m at the inlet to 147.0 m at the outlet.

It is noted that for project orientation purposes, Highway 17 within the project limits, will be assumed to run west-east. Based on the preliminary GA drawing, the Highway 17 cross-section consists of two 3.75 m wide lanes with granular shoulders ranging from 2.5 m to 2.9 m in width. A three-cable guide rail system is present along both sides of the highway in the vicinity of the culvert.

The slopes of the embankment were observed to be covered with wild grass and brush; no signs of settlement or instability were noted. The embankment slopes were graded with slopes ranging from approximately 2.0H:1V to 2.5H:1V (Horizontal:Vertical). The elevation at the centreline of the roadway was surveyed at approximately 149.7 m. The elevation of the top of the culvert was approximately 149.1 m and 149.0 m at the inlet and outlet respectively, providing for 600 to 700 mm of cover.

The site is located within a physiographic region known as the Muskrat Lake Ridges which is characterized as a steep scarp composed of Precambrian rocks overlain by a thin overburden deposit of sand and gravel. (Chapman and Putnam 1984)

The lands surrounding the culvert include, forest, brush, swampy areas and farm fields. The creek channel both upstream and downstream of the culvert is a narrow meandering channel within a swampy area. The water level was fairly low with little visible flow at the time of the field investigation (see Photos 2, 3 and 4 in Appendix E). The storm water drainage in the area is to existing culverts and ditches.

Site photographs showing the general conditions at the site, along the highway embankment and at the inlet and outlet are presented in Appendix E.

3 SITE INVESTIGATION

3.1 Field Investigation

A field investigation was carried out between June 23 and 24, 2015, and included advancing four boreholes. Due to the shallow termination depth of Borehole 601, an additional borehole, Borehole 601A, was advanced approximately 1.5 m north and west of Borehole 601.

A supplemental investigation was carried out on May 23, 2017, to further assess the very loose to loose silt and sand deposits that had been identified in the boreholes. The supplemental investigation included advancing four seismic cone penetration tests (SCPT) through the roadway, two on each side of the existing culvert. Pore pressure dissipation tests and shear wave velocity measurements were carried out in each SCPT test hole. A copy of the Seismic Cone Penetration Testing Results is provided in Appendix D.

The approximate locations and elevations of the test holes are shown on Drawing No. 1 provided in Appendix A and are summarized in Table 3-1.

Table 3-1: Test Hole Summary

Test Hole	Type	Location	Latitude (degrees)	Longitude (degrees)	Ground Surface Elevation (m)	Depth (m)
601	Borehole	Culvert Inlet	45.58692	-76.83132	148.8	3.4
601A	Borehole	Culvert Inlet	45.58694	-76.83132	148.2	2.3
602	Borehole	Highway 17 Westbound	45.58681	-76.83134	149.6	9.1
603	Borehole	Highway 17 Eastbound	45.58685	-76.83145	149.6	9.2
604	Borehole	Culvert Outlet	45.58674	-76.83147	148.6	7.6
17-01	SCPT	Highway 17 Westbound	45.58707	-76.83162	149.6	4.8
17-02	SCPT	Highway 17 Westbound	45.58688	-76.83141	149.7	6.7
17-03	SCPT	Highway 17 Eastbound	45.58679	-76.83138	149.7	8.0
17-04	SCPT	Highway 17 Eastbound	45.58660	-76.83117	149.7	7.3

As a component of our standard procedures and due diligence, Thurber contacted Ontario One Call to obtain utility locates/clearances for the intended borehole locations.

Boreholes 602 and 603 were advanced through the roadway embankment with a CME truck mounted drill rig using hollow stem augers. The inlet and outlet boreholes (Boreholes 601, 601A and 604) were advanced with portable drilling equipment using a full weight hammer, tripod and casing with washboring.

The subsurface stratigraphy encountered in the boreholes was recorded in the field by Thurber personnel. Split spoon samples were collected at regular depth intervals in the boreholes via the completion of Standard Penetration Tests (SPT), following the methods described in ASTM Standard D1586-11. All soil samples recovered from the boreholes were placed in moisture-proof containers and the samples were transported to Thurber's Ottawa geotechnical laboratory for further examination and testing. Bedrock was cored in all boreholes except Borehole 602 with NQ size coring equipment following ASTM Standard D6032-08. Bedrock core samples were stored in core boxes for transport.

A 25 mm inside diameter PVC piezometer was installed in Borehole 604 to measure the groundwater level at the site. Piezometer construction details are illustrated on the Record of Borehole sheet for Borehole 604, provided in Appendix B. The piezometer was decommissioned on December 16, 2015, after the water levels were measured.

The four SCPT test holes were advanced to refusal with a 30 Ton CPT truck mounted drill rig. Further details regarding the equipment and methodology are provided in Appendix D. The pavement structure at the SCPT test holes was pre-augered to a depth of approximately 1.5 m before advancing the cone.

The test holes without a piezometer were backfilled with a low-permeability mixture of auger cuttings and bentonite pellets in general accordance with the intent of Ontario MOE Regulation 903. Test holes advanced within the paved areas were capped with 150 mm of cold patch asphalt.

The as-drilled locations of the test holes and ground surface elevations at the borehole locations were surveyed by Thurber. The vertical datum used was a round iron bar identified on the plans provided by WSP, as a horizontal control point (HCP) and as having a geodetic elevation of 148.708 m. The location of the TBM, located at Station 18+872, is indicated on Drawing No. 1 in Appendix A.

3.2 LABORATORY TESTING

Geotechnical laboratory testing consisted of natural moisture content determination and visual identification of all soil samples in accordance with the current MTO standards. Grain size distribution analyses, and Atterberg Limits testing were also carried out on selected samples to MTO and ASTM standards. All recovered bedrock core was logged and core recoveries and RQD values were measured.

The geotechnical laboratory test results are presented on the Record of Borehole sheets in Appendix B and are illustrated on the figures in Appendix C.

Chemical analysis for determination of pH, resistivity, soluble sulphate and chloride concentrations was carried out on two soil samples. A copy of the chemical analysis results is provided in Appendix C.

4 DESCRIPTION OF SUBSURFACE CONDITIONS

4.1 Overview / General

Reference is made to the Record of Borehole sheets in Appendix B for details of the soil stratigraphy encountered in the boreholes. A stratigraphic profile for the culvert area is presented on Drawing No. 1 in Appendix A for illustrative purposes. An overall description of the stratigraphy is given in the following paragraphs; however, the factual data presented in the Record of Boreholes governs any interpretation of the site conditions.

In general, the stratigraphy within the highway adjacent to the culvert is characterized by an asphalt pavement structure, overlying sand with silt and gravel fill, overlying sand with silt and gravel overlying silt with sand, underlain by bedrock.

The bedrock profile varies considerably across the site from the south to north. The depth below existing grade to the bedrock surface ranged from 1.2 m to 2.7 m at the culvert inlet. At the culvert outlet the depth to the bedrock surface ranged from 6.7 m to 7.8 m.

More detailed descriptions of the individual strata are presented below.

4.2 Asphalt

Two boreholes were advanced through the Highway 17 pavement structure. The thickness of the asphalt was measured as 250 mm and 300 mm.

4.3 Topsoil

A layer of topsoil with a thickness of 150 mm was encountered in the outlet Borehole 604. Topsoil thickness may vary between boreholes and in other areas of the site.

4.4 Fill

A granular fill layer consisting predominantly of sand and gravel with varying amounts of silt was encountered below the surficial materials in all boreholes. The top of this layer ranges from Elevation 148.2 m to 149.4 m. The thickness of the layer ranged from 1.2 m to 2.2 m. The SPT 'N' values generally ranged from 5 to 26; indicating a loose to compact condition. One SPT conducted within the pavement structure base material resulted in 100 blows for 225 mm of penetration.

The moisture content of the samples tested ranged from 2% to 24%. The results of grain size analysis conducted on four samples of the fill material are summarized in Table 4-1 and are illustrated on Figure 1 in Appendix C.

Table 4-1: Gradation Results for Granular Fill

Soil Particles	%
Gravel	20 to 46
Sand	43 to 69
Silt and Clay	9 to 13

4.5 Sand with Silt / Silty Sand

A stratum of sand with silt to silty sand was encountered beneath the fill materials in boreholes 601, 602 and 603. The top of this layer ranges from Elevation 146.8 m to 147.3 m. The thickness of the layer ranged from 0.6 m to 2.9 m. The SPT 'N' values ranged from 2 to 21; indicating a very loose to compact condition; but typically very loose.

The moisture content for the samples tested ranged from 19% to 46%. A trace amount of organic matter was identified within this deposit in Borehole 603. The results of grain size analysis conducted on three samples of this material are summarized in Table 4-2 and are illustrated on Figure 2 in Appendix C.

Table 4-2: Gradation Results for Sand with Silt

Soil Particles	%
Gravel	0 to 19
Sand	33 to 90
Silt and Clay	9 to 48

4.6 Clay

A thin stratum of clay was encountered below the sand with silt stratum in Borehole 602. This layer was encountered at Elevation 144.7 m and has a thickness of 300 mm. The moisture content of the sample tested was 23%. Insufficient sample recovery limited the amount of laboratory testing that could be conducted on this material.

4.7 Silt with Sand

A stratum of silt with sand was encountered beneath the fill materials in Borehole 604, beneath the sand with silt stratum in Borehole 603 and below the clay layer in Borehole 602. The top of this layer ranges from Elevation 144.3 m to 146.2 m. The thickness of the layer ranged from 1.5 m to 4.3 m. The SPT 'N' values ranged from 1 to 15; indicating a very loose to compact condition; but typically very loose to loose.

The moisture content for the samples tested ranged from 13% to 27%. The results of grain size analysis conducted on five samples of this material are summarized in Table 4-3 and are illustrated on Figure 3 in Appendix C.

Table 4-3: Gradation Results for Silt with Sand

Soil Particles	%
Gravel	0 to 4
Sand	14 to 41
Silt	39 to 79
Clay	6 to 17

Atterberg Limits testing conducted on three samples of this material indicated a non-plastic silt (ML) material.

4.8 Silty Sand with Gravel Till

A glacial till layer consisting predominantly of silty sand with frequent boulders was encountered in Borehole 602. This layer was encountered at Elevation 142.8 m. Borehole 602 was terminated in this layer. An SPT 'N' value of 14 was obtained at one test depth. Below that depth, 100 blows resulted in 0 mm of penetration due to an inferred boulder.

The moisture content for the samples tested was 10% and 24%. The results of a grain size analysis test completed on a single sample of this material indicated a gravel content of 0%, a sand content of 68%, and a fines content (combined silt and clay size particles) of 32%. The results of grain size analysis are illustrated on Figure 4 in Appendix C.

4.9 Granite Bedrock

The overburden materials were underlain by a grey granite bedrock. All boreholes except Borehole 602 were advanced into bedrock by coring with NQ-size coring equipment. The bedrock profile varies considerably across the site from the north (inlet) to south (outlet).

A summary of the bedrock surface elevation is provided in Table 4-1.

Table 4-4: Bedrock Summary

Borehole	Location	Ground Surface Elevation (m)	Depth Below Existing Grade (m)	Top of Bedrock Elevation (m)
601	Culvert Inlet	148.8	2.7	146.1
601A	Culvert Inlet	148.2	1.2	146.9
603	Highway 17 Eastbound	149.6	7.8	141.8
604	Culvert Outlet	148.6	6.7	141.9

The bedrock was noted as being slightly weathered to fresh with a fracture index of 1 fracture per 0.3 m.

The total core recovery ranged from 80% to 97%, the solid core recovery ranged from 67% to 86% and the RQD values ranged from 46% to 68%. Based on the RQD values the rock mass quality ranges from poor to fair.

4.10 Groundwater Conditions

The groundwater levels were measured in the piezometer installed in Borehole 604 on December 16, 2015, at a depth of 0.93 m; corresponding to Elevation 147.7 m.

The water level in Muskrat Creek was measured at the time of Thurber's 2015 field investigation at a depth of 1.5 m below the top of the culvert at the inlet; corresponding to Elevation 147.5 m. The groundwater level in the area of the culvert is expected to reflect the water level in Muskrat Creek.

These observations are short-term readings and seasonal fluctuations of the groundwater level are to be expected. In particular, the groundwater level may be at a higher elevation after the spring snowmelt or after periods of heavy rainfall.

5 MISCELLANEOUS

Thurber staked and/or marked the borehole locations in the field and obtained utility clearances prior to drilling. Thurber surveyed the borehole locations, and determined the ground surface elevations based on contract drawings provided by WSP Canada. Ohlmann Geotechnical Services (OGS) Inc. of Almonte, Ontario supplied and operated the drilling equipment to carry out the drilling, sampling, and in-situ testing. ConeTec Investigation Ltd. supplied and operated the Seismic Cone Penetration Testing equipment. George Downing Estate Drilling Ltd. of Hawkesbury, Ontario supplied and operated the drilling equipment to carry out the pre-auguring required for SCPT testing at the site. The drilling, sampling and SCPT testing operations in the field were supervised on a full-time basis by Mr. Simon Paxton and Mr. Christopher Murray of Thurber. Laboratory testing was carried out by Thurber in its MTO-approved laboratory in Ottawa.

Overall project management and direction of the field program was provided by Paul Carnaffan, P.Eng. Interpretation of the field data and preparation of this report was completed by Kenton Power, P.Eng. The report was reviewed by Paul Carnaffan, P.Eng. and Dr. P.K. Chatterji, P.Eng., the Designated Principal Contact for MTO Foundations Projects.



Kenton C. Power, P.Eng.
Geotechnical Engineer



Paul Carnaffan, P.Eng.
Principal, Senior Geotechnical Engineer



P.K. Chatterji, P.Eng.
Review Principal, Designated MTO Contact

**FOUNDATION INVESTIGATION AND DESIGN REPORT
REPLACEMENT OF STRUCTURAL CULVERT No. 29-232/C
MUSKRAT CREEK CROSSING OF HIGHWAY 17
RENFREW COUNTY
W.P. 4113-01-01
AGREEMENT NUMBER: 4014-E-0014**

GEOCRES NUMBER: 31F-201

PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

6 GENERAL

This report presents the interpretation of the factual data obtained from a foundation investigation conducted by Thurber for the replacement of the Muskrat Creek Culvert located on Highway 17, in Renfrew, Ontario. Geotechnical recommendations are provided to assist the design team in designing a suitable foundation for the proposed replacement culvert.

This foundation investigation and design report with the interpretation and recommendations are intended for the use of the Ministry of Transportation, and shall not be used or relied upon for any other purposes or by any other parties including the construction or design-build contractor. Contractors must make their own interpretation based on the factual data in Part 1 of the report. Where comments are made on construction, they are provided only in order to highlight those aspects which could affect the design of the project. Contractors must make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods and scheduling.

No previous foundation investigation information for the subject culvert was available. A Preliminary General Arrangement (GA) drawing dated December 2017, and base plan mapping were provided by WSP for the preparation of this report. A copy of the Preliminary GA drawing is provided in Appendix A.

The following sections address geotechnical recommendations for the replacement of the existing Muskrat Creek Culvert. The discussions and recommendations presented in this report are based on the information provided by WSP and on the factual data obtained during the course of this investigation.

6.1 Proposed Structure

Based on the December 2017 GA drawing provided by WSP, the existing culvert is to be replaced with a 25 m long, closed bottom concrete culvert with a span of 4.0 m, and a height of 2.4 m. The new culvert is to be installed on a new alignment to the west of the existing culvert. The centreline of the new culvert alignment is to be offset from the existing culvert by approximately 5.6 m and 5.3 m at the outlet and inlet, respectively. The east edge of the new box culvert will be offset approximately 1.1 to 0.8 m from the west edge of the existing culvert. It is understood that neither retaining walls or wingwalls are proposed for this project.

The top of streambed elevation will be at approximately Elevation 146.8 m at the upstream end and Elevation 146.7 m at the downstream end.

No changes to the profile of Highway 17 above the culvert are proposed, however, temporary embankment widenings may be constructed to allow for staged construction while maintaining two lanes of traffic.

It is understood that creek flow will be maintained through the existing culvert during installation of the new box culvert. The preliminary plan includes abandoning the existing culvert by leaving it in place and filling it with grout or concrete after construction of the new culvert is complete.

6.2 Applicable Codes and Design Considerations

The geotechnical assessment presented below has been prepared based on the available data regarding the proposed foundations and existing ground conditions and in accordance with the Canadian Highway Bridge Design Code, version CSA S6-14, (CHBDC).

In accordance with CHBDC, the analysis and design of the structures takes into consideration the importance of the structures and the consequence associated with exceeding limit states. The importance category and consequence classification are defined by the Regulatory Authority, which in this case is the Ministry of Transportation, Ontario (MTO).

It is understood that the culvert is being designed to the Major Route seismic importance category due to its location on Highway 17.

It is also understood that this culvert has been assigned Typical Consequence Classification, in accordance with Section 6.5.1 of the CHBDC. Accordingly, a consequence factor (Ψ) of 1.0, as per Table 6.1 of the CHBDC, has been used in assessing factored geotechnical resistances. If the consequence classification changes, the geotechnical assessment will need to be reviewed and revised.

The estimated frost penetration depth at this site is 1.9 m as per OPSD 3090.101.

7 SEISMIC CONSIDERATIONS

7.1 Spectral and Peak Acceleration Hazard Values

The seismic hazard data for the CHBDC is based on the fifth-generation seismic model developed by the Geological Survey of Canada (GSC). Seismic hazard data for this site has been obtained from the GSC's seismic hazard calculator. The data includes peak ground acceleration (PGA), peak ground velocity (PGV), and the 5% damped spectral response acceleration values ($S_a(T)$) for the reference ground condition (Site Class C) for a range of periods (T) and for a range of return periods including the 475-year, 975-year and 2475-year events. The GSC seismic hazard calculation data sheet for this site is presented in Appendix G.

7.2 Seismic Liquefaction Assessment

The results of the 2015 borehole investigation indicated the presence of very loose to loose silt and sand deposits which were assessed to be liquefiable. The supplemental investigation completed in 2017 included SCPT testing to minimize the potential for ground disturbance inherent with SPT testing and to allow for a more rigorous assessment of the potential for liquefaction at the site.

The raw SCPT data obtained during the field investigation was processed by Thurber using GeoLogismiki's CPTe-IT and CLiq software. The interpreted results were compared with previous boreholes completed near the culvert. A comparison of the CPT results is illustrated on Figure 2 in Appendix G. Based on the low SPT 'N' values in the sand and silt, CPT I_c values less than 2.6 and index testing on disturbed soil samples, the loose sand and non-plastic silt has the potential to experience liquefaction.

Based on a design PGA of 0.11 g for the 1:475 earthquake, the culvert foundation is not anticipated to liquefy. However, for the design PGA of 0.18 g for the 1:975 earthquake, the full depth of loose sand and silt below the culvert is anticipated to liquefy. A similar extent of liquefaction would be expected for the 1:2475 earthquake. Outputs of the cyclic stress ratio and cyclic resistance ratio for the 1:475, 1:975 and 1:2475 earthquakes for SCPT Test holes 17-02 and 17-03, which are closest to the culvert, are shown on Figure 1 provided in Appendix G.

Seismic stability analyses were completed using GeoStudio 2012 Slope/W software. The post-liquefaction factor of safety was checked for flow side failures towards the outlet (south) and inlet (north) side of the culvert in the 1:975 and 1:2475 earthquakes. The results of the analyses towards the south and north are shown on Figures 3 and 4 provided in Appendix G. The red shading illustrated on Figures 3 and 4 represent possible regions where the estimated failure surface has a factor of safety of 1.0 or less and suggests these areas have a high potential for flow slide failure in the transverse direction (significant lateral deformation under gravity loading). It should be noted that the embankment away from the culvert will also likely experience a similar pattern of flow slide failure in the transverse direction.

The culvert and existing roadway will experience settlement and lateral deformation following a seismic event. The post-seismic settlement profile at this site will be influenced by a combination of post-liquefaction vertical settlement, shear-induced settlement due to embankment failure and varying depth to firm stratum along the culvert profile. The estimated settlement is summarized in Table 7-1.

Table 7-1: Estimated Post-Liquefaction Settlement Profile

Earthquake Event	Inlet (North) (mm)	Below Westbound Lane (SCPT17-02) (mm)	Below Eastbound Lane (SCPT17-03) (mm)	Outlet (South) (mm)
1:475	< 25	< 25	< 50	< 50
1:975	< 25	< 50	100	175
1:2475	< 25	75	150	225

The culvert crossing and adjacent embankment could experience a flow slide failure following a 1:975 or 1:2475 year earthquake event. The magnitude of lateral deformation could range from 1.0 m to 5.0 m or more. Local variations in fill height and adjacent topography will influence the magnitude of deformations. If the culvert is designed to accommodate this magnitude of movement, there are still differential movements to consider due to flow slide failure of the embankment immediately adjacent to the culvert.

7.3 CHBDC Seismic Site Classification

In accordance with the CHBDC, sites with liquefiable soils are designated as Site Class F. However, Section 4.4.3.3 of the CHBDC allows that the Site Class and corresponding values of $F(T)$ for structures with a fundamental period of ≤ 0.5 seconds that are built on liquefiable soils, may be determined by assuming that the soils are not liquefiable. It is understood that the

fundamental period of the culvert is expected to be less than 0.5 seconds, and therefore this section of the code applies. Based on the shear wave velocities measured during SCPT testing carried out at the site, the culvert may be designed using Seismic Site Class E.

Site coefficients in Tables 4.2 to 4.9 of the CHBDC can be applied to the Site Class C hazard values to develop the design spectral envelope for Seismic Site Class E. For the culvert, the design peak ground acceleration for a 1:475, 1:975 and 1:2475 year return period earthquakes are approximately 0.11g, 0.18g and 0.24g, respectively.

8 GEOTECHNICAL ASSESSMENT

8.1 Design Considerations

Design considerations from a geotechnical perspective include, but are not limited to, the following:

- The ground conditions beneath the culvert alignment include very loose to loose sand and silt deposits that are susceptible to liquefaction under the design earthquake event. Liquefaction of the foundation soils can lead to large deformation of the foundation soils (vertical and lateral) and loss of bearing resistance. In addition, the very loose to loose sand and silty sand deposits offer low bearing resistance for foundations under static conditions.
- The culvert has minimal cover (approximately 600 to 700 mm).
- The elevation of the bedrock surface is highly variable and is above the founding elevation near the inlet. Bedrock excavation may be required if the culvert is widened or constructed on a new alignment. The variable/sloping rock surface may also present challenges for installation of temporary protection systems.
- A temporary flow passage system and dewatering of work excavations will be required. The proximity of the new culvert to the existing culvert must be considered to ensure that the excavations and loads from the culverts do not adversely impact the other structure. A temporary protection system may be required. If the existing culvert is to be used as a component of a temporary flow passage system or temporary protection system, it must be confirmed that the structure can support unbalanced earth pressures and/or hydrostatic pressures that may develop.
- The Ministry requires that two lanes of traffic be maintained throughout construction, thereby requiring a temporary widening, even if a temporary protection system is used along the centreline of the highway.

Foundation alternatives to address both the static and seismic design requirements including liquefaction are discussed and evaluated in Section 8.2 and 8.3 and the recommended alternative from a foundation engineering perspective is presented in Section 8.4.

8.2 Foundation Design Alternatives – Liquefaction Mitigation

The following three alternatives were considered to mitigate the seismic liquefaction issue:

- Excavate to remove liquefiable soils and replace with engineered fill, followed by construction of a conventional culvert structure
- Carry out ground improvement to stabilize the liquefiable soils, followed by construction of a conventional culvert structure

- Leaving the existing liquefiable soils in place and designing the culvert to withstand post-seismic kinematic loads. This alternative could include a rigid cast-in-place box culvert or a culvert supported on deep foundations, however, given the shallow but variable depth to rock, a cast-in-place box culvert is considered to be more practical and cost effective for this site.

These foundation alternatives have been evaluated from a geotechnical perspective in terms of their respective advantages, disadvantages, risks and consequences. The evaluation is summarized in the tables provided in Appendix F.

8.3 Culvert Type Alternatives

Common culvert and foundation types are listed below and a comparison of these alternatives from a foundation perspective, based on their respective advantages and disadvantages is outlined below, and summarized in a table provided in Appendix F.

8.3.1 Circular Pipes

From a foundation engineering perspective, circular pipes installed with appropriate granular bedding native are feasible for static design conditions. However, it is understood that numerous circular pipes on new alignments would be required to provide the required hydraulic opening. Also, it is unlikely that the circular pipes could be designed to withstand the anticipated kinematic loads, therefore the use of circular pipes would require full removal and replacement of the liquefiable soils or ground improvement.

8.3.2 Open Bottom Culvert

An open bottom culvert was considered for this project; however, the sand and silt subgrade offers relatively low bearing resistance which is insufficient based on the proposed size of the structure at this site. This option would only be feasible if ground improvement or full removal and replacement of the liquefiable soils was carried out.

8.3.3 Closed Bottom (Box) Culvert

From a geotechnical perspective, the replacement could be achieved with a closed bottom concrete box culvert. A closed box culvert offers several advantages including spreading the static load over a wider area. In addition, a closed box can more easily be designed to resist kinematic loading associated with liquefaction.

Based on a substrate with a top elevation of approximately 146.7 m and a thickness of 350 mm and allowing for a 260 mm thick concrete base, the base of excavation is expected to be around Elevation 145.6 m to 146.0 m (depending on whether a granular pad or a concrete working slab is provided) at which elevation the subgrade would be in the native loose sandy silt to silty sand. Bedrock excavation may be required at the culvert inlet (north end of the culvert) to reach the design culvert invert elevation.

8.3.4 Pre-cast vs Cast-in-Place Concrete Culverts

A cast-in-place culvert can be constructed as a single, ridged unit to better resist the post-seismic kinematic loads than a series of pre-cast units. Also, a cast-in-place culvert is less prone to disturbance during the removal of temporary protection systems. The use of pre-cast units will

generally allow for quicker installation, possibly reducing dewatering requirements and the overall construction schedule. Larger cranes are likely required for installation of large span pre-cast units which may impact the required construction staging zone.

8.4 Recommended Approach for Culvert Replacement

Based on the evaluations presented above and in Appendix F, the recommended design approach from a foundation engineering perspective is to leave the existing liquefiable soils in place and to design the replacement culvert to withstand post-seismic kinematic loads which includes settlement and lateral movement of the embankments. The recommended culvert type in order to resist the post-seismic kinematic loads and based on the low static bearing resistance is a cast-in-place concrete box structure.

9 FOUNDATION DESIGN RECOMMENDATIONS

9.1 Culvert Foundation Bearing Resistances

Based on the December 2017 Preliminary GA drawing, the design top of substrate is noted as between Elevations 146.7 m and 147.8 m with a minimum thickness of 350 mm. Assuming a culvert base thickness of 260 mm, the culvert will be founded at approximately Elevation 146.0 m. The native subgrade within the footprint of most of the culvert is expected to consist of undisturbed native loose sandy silt to silty sand. Bedrock excavation may be required at the culvert inlet (north end of the culvert) to reach the design culvert invert elevation.

A cast-in-place, concrete box culvert between 4.5 m and 5.5 m in width, founded at Elevation 146.0 m on a concrete mud slab or a granular pad at least 0.5 m thick, can be designed with the following factored geotechnical resistances:

- Factored geotechnical resistance at ULS; 150 kPa
- Factored geotechnical resistance at SLS; 100 kPa

The factored geotechnical resistances include the following factors:

- The factored geotechnical resistance values at SLS provided above correspond to the stress increase relative to current site conditions that will result in 25 mm of total settlement.
- Consequence factor (Ψ) of 1.0
- Geotechnical resistance factors (CHBDC Table 6.2):
 - Bearing (ULS), $\phi_{gu} = 0.5$; (static analysis; typical degree of understanding)
 - Settlement (SLS), $\phi_{gs} = 0.8$; (static analysis; typical degree of understanding)

The structural design of the culvert should consider differential settlement across the culvert. At the inlet where the bedrock is the anticipated founding material settlement is considered negligible under the anticipated loadings. Based on the SLS bearing resistance provided above, a maximum of 25 mm of differential settlement should be anticipated between the portions of the culvert founded on the native soils and those founded on Bedrock.

The geotechnical resistances provided in Section 9.1 are for vertical concentric loading and will need to be adjusted for the effects of inclined or eccentric loading, if applicable, as illustrated in the CHBDC Clause 6.10.3 and Clause 6.10.4.

Resistance to lateral forces through sliding resistance should be evaluated using the following unfactored coefficients of friction:

- Cast-in-place concrete on native sand 0.45
- Cast-in-place concrete on Engineered Fill 0.55

9.2 Subgrade Preparation, Culvert Bedding and Backfilling

Excavation and backfilling for installation of the new culvert should be carried out in accordance OPSS 902 and MTO Special Provision (SP) No. 109S12, Amendment to OPSS 902, March 2018.

The creek water level was observed at Elevation 147.5 m during the field investigation. As such, the base of the excavation would range from 1.5 m to 2.0 m below the measured creek level. Please refer to Section 10.3 for additional comments on groundwater and surface water control.

The native subgrade within the footprint of the culvert is expected to consist mostly of native very loose to loose silt with sand to silty sand. However, bedrock excavation is anticipated to be required to achieve the design invert elevation near the inlet.

The subgrade preparation should include construction of a transition to reduce the potential for non-uniform and abrupt differential settlement (i.e. hard point effect) between the possible bedrock and soils. Suggested wording for an NSSP to alert the Contractor to the requirement for bedrock excavation and details of the bedrock condition at the site has been provided in Appendix H.

The very loose to loose silt and sand materials will be easily disturbed when saturated, subjected to construction or personal traffic, freeze thaw actions, ingress or ponding water. One option to protect the exposed subgrade is to ensure adequate dewatering and to place geotextile over the exposed native subgrade and constructing a 0.5 m thick granular pad over the entire exposed footprint prior to constructing the culvert. The geotextile should consist of a non-woven, Class II geotextile in accordance with OPSS 1860.

Alternatively, the exposed subgrade could be covered with a 100 mm thick concrete working slab. After the concrete for the working slab has set, the culvert could then be constructed directly on the working slab without the need for a granular pad or bedding material. Suggested wording for an NSSP to alert the Contractor to the requirement for a working slab has been provided in Appendix H.

Backfill for the culvert must consist of granular material conforming to OPSS.PROV 1010 Granular A or Granular B Type II material specifications. Heavy compaction equipment, used adjacent to the culvert, must be restricted in accordance with OPSS.PROV 501. Care must be exercised when compacting the fill adjacent to and above the culvert in order not to damage the culvert.

It is recommended that the backfill detailing of OPSD 803.010 be utilized with a frost penetration line below the top of the culvert. The frost treatment depth, k , should be set at 1.9 m. The depth of road bed granulars, d , should be set at 0.790 m.

9.3 Lateral Earth Pressures

Lateral earth pressures parameters provided in Table 9-1 in the section below are based on the assumption that the backfill is fully drained so that there are no unbalanced hydrostatic pressures. If adequate drainage cannot be confirmed, the potential for buildup of hydrostatic pressures should be considered in design.

9.3.1 Static Lateral Earth Pressure Coefficients

Lateral earth pressures acting on structures should be computed in accordance with the CHBDC but generally are given by the expression:

$$P_h = K^*(\gamma h + q)$$

where:

P_h = horizontal pressure on the wall (kPa)

K = earth pressure coefficient

γ = unit weight of retained soil (kN/m³)

h = depth below top of fill where pressure is computed (m)

q = value of any surcharge (kPa)

A lateral earth pressure due to backfill compaction should be added to the calculated lateral earth pressure in accordance with Clause 6.12.3 of the CHBDC.

The recommended lateral earth pressure parameters for use in the design for a horizontal back-slope are provided in Table 9-1.

Table 9-1: Static Lateral Earth Pressure Coefficient

Parameter	OPSS Granular A & B Type II	Existing Fill	Sandy Silt / Silty Sand
Soil Unit Weight, kN/m ³ , γ	21.0	20.0	18.0
Angle of Internal Friction, ϕ	35°	33°	28°
Coefficient of at Rest Earth Pressure, K_o (Restrained Wall)	0.43	0.46	0.53
Coefficient of Active Earth Pressure, K_a (Unrestrained Wall)	0.27	0.29	0.36
Coefficient of Passive Earth Pressure, K_p (Unrestrained Wall)	3.69	3.39	2.77

If there is a sloped embankment or backfill above the protection system, the earth pressure parameters must be revised accordingly.

For rigid structures, it is recommended that at-rest horizontal lateral earth pressures be used for design. Active pressures should be used for the design of unrestrained walls.

A lateral pressure due to backfill compaction should be added to the calculated lateral earth pressure in accordance with Section 6.12.3 of the CHBDC.

9.3.2 Seismic Lateral Earth Pressure Parameters

Due to the presence of liquefiable soils, seismic forces acting on the culvert are expected to be greater than those derived using pseudo-static methods (e.g. Mononobe-Okabe method). The detailed design should consider loading associated with kinematic loads and the liquefaction induced ground deformation.

9.4 Embankment Design and Reinstatement

Due to the limited cover, embankment reinstatement for the new culvert will consist of structure backfill and the reinstated pavement structure. Please refer to the pavement design memo for the recommended pavement structure.

The existing embankments have slopes ranging from approximately 2.0H:1V to 2.5H:1V and exhibit no signs of instability. The embankments should be reinstated to match the adjacent slopes.

The embankment construction should be carried out in accordance with OPSS.PROV 206. Embankment fill, beyond the limits of structure backfill, should consist of Select Subgrade (SSM) material or better in compliance with OPSS.PROV 1010. The embankment constructed with side slopes at 2H:1V or flatter are considered stable under static conditions. The fill material should be placed and compacted in accordance with OPSS.PROV 501.

9.5 Cement Type and Corrosion Potential

A sample of the native soil was submitted to Paracel Laboratories in Ottawa, Ontario for analysis of pH, water soluble sulphate and chloride concentrations, and resistivity. The analysis results are summarized in the Table 9-2 and a copy of the test results is provided in Appendix C.

Table 9-2: Results of Chemical Analysis

Borehole	Sample	Depth (m)	pH	Resistivity (Ohm-cm)	Chloride (µg/g)	Sulphate (µg/g)
601	SS4	2.2	7.7	3100	70	112

The concentration of soluble sulphate provides an indication of the degree of sulphate attack that is expected for concrete in contact with soil and groundwater at the site. The soluble sulphate results in Table 9-2 were compared with the parameters in Table 3 of the Canadian Standards Association Standards A23.1-14 (CSA A23.1) and indicate a low degree of sulphate attack potential on concrete structures at this site.

The pH, resistivity and chloride concentration provide an indication of the degree of corrosiveness of the sub-surface environment. The test results provided in Table 9-2 were compared with Table 3.2 of the MTO Gravity Pipe Design Guideline and generally indicate a moderately corrosive environment. The test results provided in Table 9-2 may be used to aid in the selection of coatings and corrosion protection systems for buried steel objects.

10 CONSTRUCTION CONSIDERATIONS

10.1 Excavations

Excavation for the culvert replacement must be carried out in accordance with OPSS 902.

All excavations must be conducted in accordance with the requirements of the Occupational Health & Safety Act & Regulations (OHSA) for Construction Projects. The fills at the site should be classified as Type 3 and the very loose to loose native sand and silt materials located below the level of the groundwater and/or the water level in the creek should be considered as Type 4 soils in accordance with OHSA. However, as indicated in the OHSA, if an excavation contains more than one type of soil, the soil type for the excavation shall be classified as the type with the highest number among the soil types present within the excavation. In accordance with OHSA unsupported excavations made in Type 4 soils must have side slopes no steeper than 3H:1V from the base of the excavation.

The contractor must consider the potential for unbalanced lateral earth pressures on the existing culvert while excavating for the new culvert and must protect the existing culvert from damage and movement. If required, a temporary protection system may be used to protect the existing culvert during construction. The support system should be designed and installed in accordance with OPSS.PROV 539. The design of the excavation support system is the responsibility of the Contractor. The support system must be design by a Professional Engineer experienced in such design. Typical lateral earth pressure coefficients are provided in Table 9-1 for the design of the support.

At locations where there are space restrictions or where a slope has to be retained, the excavations will need to be carried out within a protection system. Further discussion is presented in Section 10.2.

Subgrade preparation and placement of culvert bedding must be carried out in the dry. Further discussion regarding dewatering is provided in Section 10.3.

Selection of the equipment and methodology to excavate and prepare the founding surface is the responsibility of the Contractor.

10.2 Temporary Protection Systems

It is anticipated that the culvert replacement will be carried out in two stages with both a temporary platform widening and a temporary protection system. Where required, Temporary Protection Systems to support the existing highway platform should be provided in accordance with OPSS.PROV 539 and designed for Performance Level 2. A more stringent performance level may apply if the foundations of existing structures must be protected. All protection systems should be designed by a Professional Engineer experienced in such designs. Typical lateral earth pressure coefficients are provided in Table 9-1.

The design of temporary protection systems is the responsibility of the Contractor. The designer of the temporary protection system must ensure the penetration depth is sufficient to provide base fixity and lateral stability. The design must incorporate traffic loading and surcharge loading due to construction equipment and their operations and shall consider the slope of embankments above the top of the protection system and location of existing utilities and trenches.

The variable rock surface may present challenges for protection systems. Suggested wording for an NSSP has been provided in Appendix H to alert the Contractor that the bedrock profile varies considerably across the site and that temporary protection system will be installed in ground conditions that include sloping bedrock.

10.3 Dewatering

The Contractor must be prepared to control the groundwater and surface water flow at the site to permit the proposed culvert replacement to be constructed in a dry and stable excavation. The groundwater level for the site at the time of the proposed replacement should be taken as the water level in the creek. It is recommended that the replacement be conducted during a drier season such as after the spring freshet or prior to the fall season.

It is understood that the existing culvert will remain operational and will serve as a temporary flow passage for Muskrat Creek during construction of the new box culvert. It is understood that the existing culvert is to be abandoned in place and decommissioned by filling with concrete after construction of the new culvert is complete.

Excavations below the groundwater level are anticipated for constructing the box culvert. A cofferdam with pumping from sumps may be required to control inflow of water into the excavation prepare the subgrade and to construct the footings in the dry. Dewatering and surface water diversion must remain operational and effective until the culvert is replaced.

The design of any dewatering system that may be required is the responsibility of the Contractor. The Contract Documents must alert them to this responsibility and to design the system in accordance with SP No. FOUN0003.

The Dewatering Systems Designer Fill-in information for SP No. FOUN0003 are provided in the following table.

Design Storm Return Period	Preconstruction Survey Distance
*	**
Where required, fill-in information will be provided in the WSP's Draft Hydrogeological Assessment in Support of an Environmental Activity and Sector Registry (EASR) for Culvert Replacement Work at Muskrat Creek dated March 2018	N/A

In accordance with Section 902.04 of SP FOUN0003, the dewatering system is to be designed in accordance with OPSS.PROV 517 and SP 517F01; Amendment to OPSS 517, July 2017.

The Table A Fill-ins for SP 517F01 are as provided below.

IDF Curve Location		Latitude: 45.58687°		Longitude: -76.83145°		
Temporary Flow Passage Systems						
Site Name / Station Reference	Minimum Return Period (Years)	Return Period Flow Estimates (m³/s)				Design Engineer Requirements (Note 1)
		2 Year	5 Year	10 Year	25 Year	
**	***	****	****	****	****	*****
Site 29-232/C Muskrat Creek Culvert Crossing of Highway 17, Approximate Station 18+950	Where required, fill-in information will be provided in the WSP's Draft Hydrogeological Assessment in Support of an Environmental Activity and Sector Registry (EASR) for Culvert Replacement Work at Muskrat Creek dated March 2018					
Dewatering Systems						
Site Name / Station Reference		Preconstruction Survey Distance (Note 2) (m)		Design Engineer Requirements (Note 1)		
**		*****		*****		
Site 29-232/C Muskrat Creek Culvert Crossing of Highway 17, Approximate Station 18+950		N/A		No		
Note: 1. "Yes" means the design Engineer and design-checking Engineer shall have a minimum of 5 years of experience in designing systems of similar nature and scope to the required work. "No" means a minimum experience level is not required for the design Engineer and design-checking Engineer. 2. "N/A" indicates a preconstruction survey is not required.						

The groundwater level will fluctuate and the minimum groundwater elevation for the site at the time of the proposed culvert replacement should be taken as the water level in the creek at the time of construction as defined by SP 517F01. Excavation below the groundwater level to construct the culvert foundation will be required and excavation below the groundwater level without prior dewatering is not recommended since the inflow of groundwater will cause base heave/boiling and sloughing of the foundation soil below the water level, making it difficult to maintain a dry, sound base on which to work.

The preliminary design allows for creek flow to be maintained through the existing culvert during construction of the new culvert, followed by realignment of the creek. Cofferdams may be required to prevent the creek from spilling into the adjacent excavation for the new culvert and during creek realignment. Further assessment of dewatering requirements and the need for a PTTW should be carried out by specialists experienced in this field. Reference should be made to WSP's Draft Hydrogeological Assessment in Support of an Environmental Activity and Sector Registry (EASR) for Culvert Replacement Work at Muskrat Creek dated March 2018, with regards to volume of water expected to be withdrawn for the excavations for the culvert replacement.

10.4 Erosion Protection

Slope protection and drainage measures will be required to ensure the long-term surficial stability of the embankment slopes. The contractor should provide silt fences and erosion control blankets, as required, throughout the duration of the construction to prevent silt/sediments from running off the site as per OPSS 805.

Erosion protection should be provided at the culvert inlet and outlet areas. Details on the erosion control analyse and protection measures proposed for this project are illustrated on the Preliminary GA drawing. Further reference should be made to WSP's Hydrology and Hydraulic Assessment Report, dated March 2018. If the Contractor proposes design changes to erosion protection measures they must consider hydrologic and hydraulic factors presented in WSP's report and should be carried out by specialists experienced in this field. The results of the final erosion control analyse should be reviewed by the foundation engineer to ensure that the foundation design and related earthworks are in accordance with the recommendations provided.

Typically, rock protection should be provided over all surfaces with which culvert water is likely to be in contact. Treatment at the outlets should be in accordance with OPSS 810.010. A vegetation cover should be established on all other exposed earth surfaces to protect against surficial erosion in general accordance with OPSS.PROV 804.

It is recommended that a clay seal be used to minimize the potential for erosion near the inlet area. The clay seal should extend a minimum of 0.3 m above the high-water level and laterally for the width of the granular material, and have a minimum thickness of 0.5 m. The material requirements should be in accordance with OPSS.PROV 1205. A geosynthetic clay liner may be used as a clay seal.

10.5 Construction Concerns

The planned construction methodology includes an open cut excavation for the installation of a new culvert. Potential construction concerns include, but are not necessarily limited to, the following:

- Construction will extend below the water level in the creek. An adequate and effective surface water management and dewatering plan must be implemented to construct the replacement culvert, to place and compact the engineering fill subgrade, and bedding below the culvert in the dry.
- Confirmation that the backfill is adequately placed and compacted to specifications.
- Increased difficulty with the installation of protection systems should the be required should be anticipated due to the relatively shallow depth to bedrock at the culvert inlet.
- Depending on the excavation and dewatering procedures employed by the Contractor, unbalanced earth and hydrostatic pressure may develop on the existing culvert walls and/or foundations while excavations for the new culvert are carried out. These forces must be taken into consideration in the design of the excavation/protection systems and dewatering procedures employed by the Contractor.
- The subgrade at the base of the excavation will be easily disturbed and should be protected throughout the construction period.

The successful performance of the construction of this structure will depend largely upon good workmanship and quality control during construction. Observation of the excavation and backfilling operations will be required as per MTO SP No. 109S12, amendment to OPSS 902 during construction to confirm that the foundation recommendations are correctly implemented and material specifications are met.

11 CLOSURE

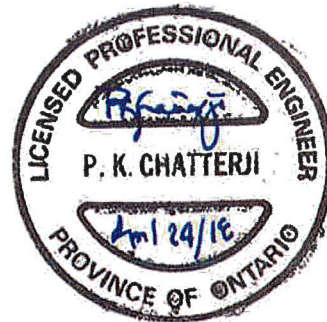
Overall project management and direction of the field program was provided by Paul Carnaffan, P.Eng. Interpretation of the field data and preparation of this report was completed by Kenton Power, P.Eng. The report was reviewed by Paul Carnaffan, P.Eng. and Dr. P.K. Chatterji, P.Eng., the Designated Principal Contact for MTO Foundations Projects.



Kenton C. Power, P.Eng.
Geotechnical Engineer



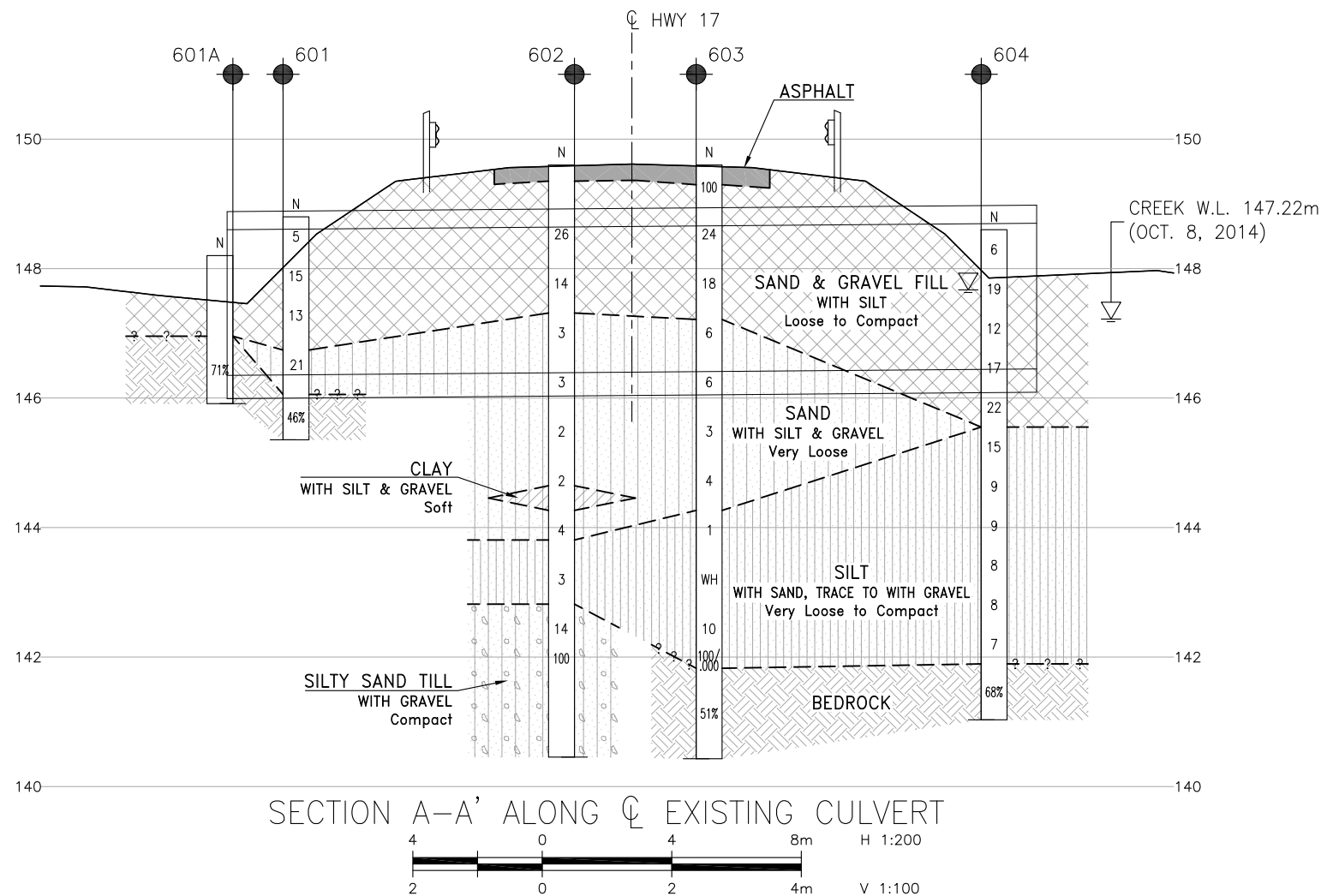
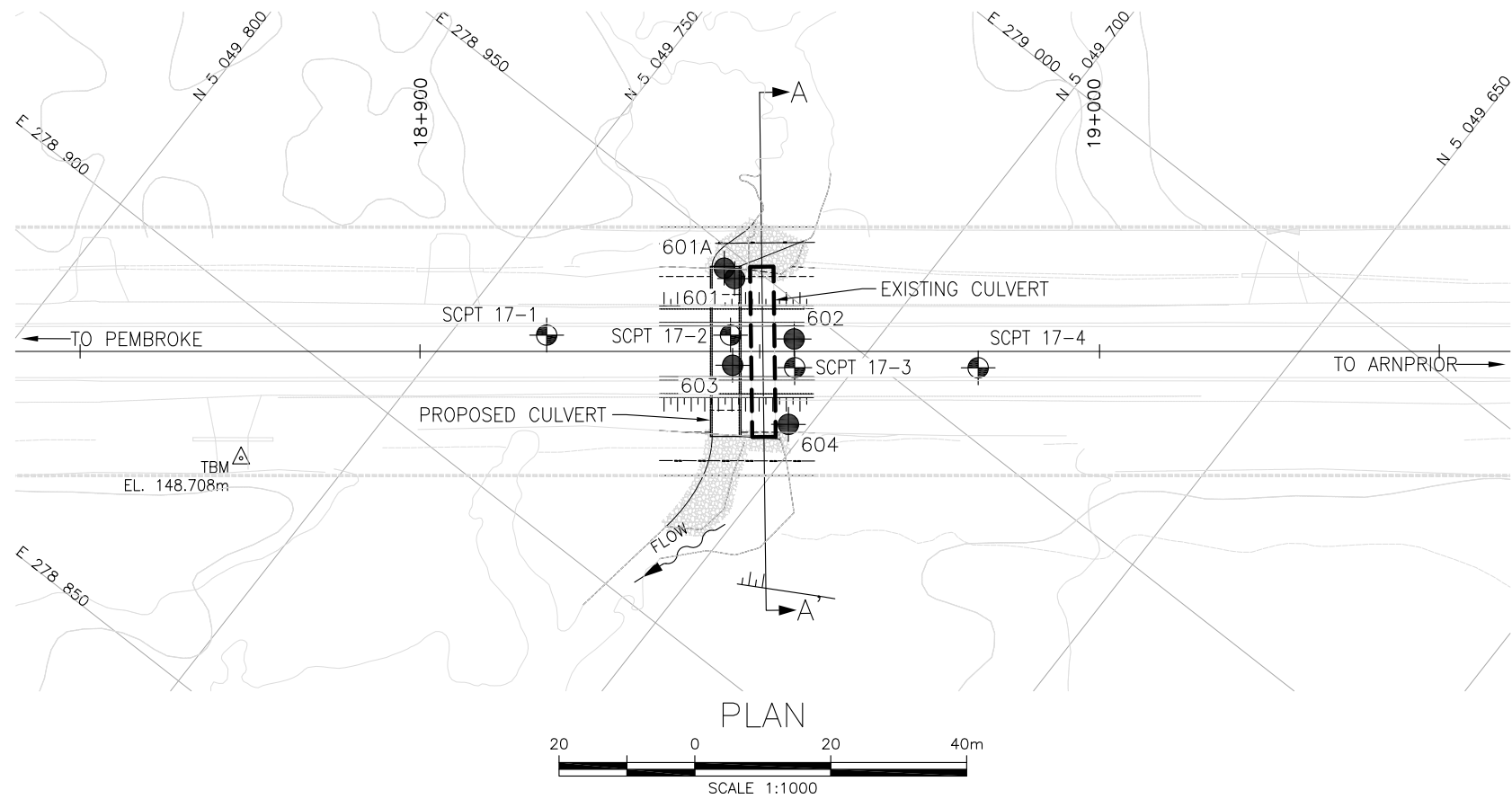
Paul Carnaffan, P.Eng.
Principal, Senior Geotechnical Engineer



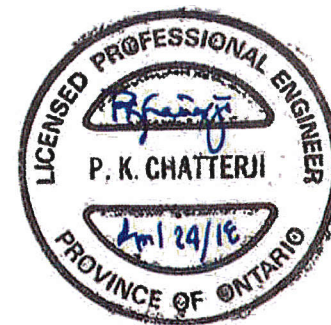
P.K. Chatterji, P.Eng.
Review Principal, Designated MTO Contact

APPENDIX A

BOREHOLE LOCATIONS AND SOIL STRATA DRAWINGS PRELIMINARY GENERAL ARRANGEMENT DRAWING



METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

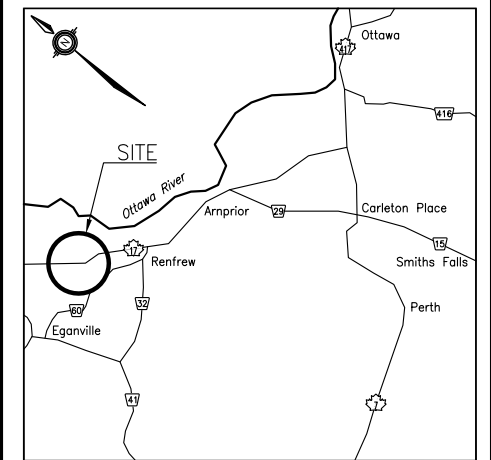


CONT No
WP No 4113-01-01

HIGHWAY 17
MUSKRAT CREEK
CULVERT REPLACEMENT
BOREHOLE LOCATIONS AND SOIL STRATA



THURBER ENGINEERING LTD.



KEYPLAN

LEGEND

●	Borehole
⊙	SCPT Hole
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
▽	Water Level
⊕	Head Artesian Water
⊕	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
601	148.800	5049722.054	278947.074
601A	148.200	5049724.194	278947.344
602	149.600	5049709.674	278945.473
603	149.600	5049714.416	278936.843
604	148.600	5049702.590	278935.053
SCPT 17-1	149.6	5 049 738.7	278 923.5
SCPT 17-2	149.7	5 049 717.4	278 940.2
SCPT 17-3	149.7	5 049 707.0	278 942.2
SCPT 17-4	149.7	5 049 685.7	278 958.8

-NOTES-

- The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- This drawing is for subsurface information only. Surface details and features are for conceptual illustration.
- Borehole locations are shown in MTM Zone 9 coordinates.

GEOCRES No. 31F-201

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	KP	CHK -	CODE
DRAWN	MFA	CHK KP	SITE 29-232/C/STRUCT
			LOAD
			DATE
			APR 2018
			DWG 1

APPENDIX B
RECORD OF BOREHOLE SHEETS



SYMBOLS, ABBREVIATIONS AND TERMS USED ON TEST HOLE RECORDS

TERMINOLOGY DESCRIBING COMMON SOIL GENESIS

Topsoil	mixture of soil and humus capable of supporting vegetative growth
Peat	mixture of fragments of decayed organic matter
Till	unstratified glacial deposit which may include particles ranging in sizes from clay to boulder
Fill	material below the surface identified as placed by humans (excluding buried services)

TERMINOLOGY DESCRIBING SOIL STRUCTURE:

Desiccated	having visible signs of weathering by oxidization of clay materials, shrinkage cracks, etc.
Fissured	having cracks, and hence a blocky structure
Varved	composed of alternating layers of silt and clay
Stratified	composed of alternating successions of different soil types, e.g. silt and sand
Layer	> 75 mm in thickness
Seam	2 mm to 75 mm in thickness
Parting	< 2 mm in thickness

RECOVERY:

For soil samples, the recovery is recorded as the length of the soil sample recovered.

N-VALUE:

Numbers in this column are the field results of the Standard Penetration Test: the number of blows of a 63.5 kg hammer falling 0.76 m, required to drive a 50 mm O.D. split spoon sampler 0.3 m into undisturbed soil. For samples where insufficient penetration was achieved and N-value cannot be presented, the number of blows are reported over the sampler penetration in millimetres (e.g. 50/75).

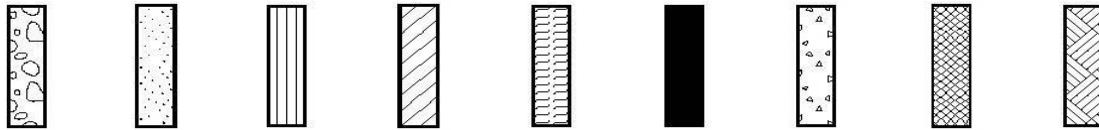
DYNAMIC CONE PENETRATION TEST (DCPT):

Dynamic cone penetration tests are performed using a standard 60 degree apex cone connected to an "A" size drill rods with the same standard fall height and weight as the Standard Penetration Test. The DCPT value is the number of blows of the hammer required to drive the cone 0.3 m into the soil. The DCPT is used as a probe to assess soil variability.



STRATA PLOT:

Strata plots symbolize the soil and bedrock description. They are combinations of the following basic symbols. The dimensions within the strata symbols are not indicative of the particle size, layer thickness, etc.



Boulders
Cobbles
Gravel Sand Silt Clay Organics Asphalt Concrete Fill Bedrock

TEXTURING CLASSIFICATION OF SOILS

Classification	Particle Size
Boulders	Greater than 200 mm
Cobbles	75 – 200 mm
Gravel	4.75 – 75 mm
Sand	0.075 – 4.75 mm
Silt	0.002 – 0.075 mm
Clay	Less than 0.002 mm

TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

Descriptive Term	Undrained Shear Strength (kPa)
Very Soft	12 or less
Soft	12 – 25
Firm	25 – 50
Stiff	50 – 100
Very Stiff	100 – 200
Hard	Greater than 200

NOTE: Clay sensitivity is defined as the ratio of the undisturbed strength over the remolded strength.

SAMPLE TYPES

SS	Split spoon samples
ST	Shelby tube or thin wall tube
DP	Direct push sample
PS	Piston sample
BS	Bulk sample
WS	Wash sample
HQ, NQ, BQ etc.	Rock core sample obtained with the use of standard size diamond coring equipment

TERMS DESCRIBING CONSISTENCY (COHESIONLESS SOILS ONLY)

Descriptive Term	SPT "N" Value
Very Loose	Less than 4
Loose	4 – 10
Compact	10 – 30
Dense	30 – 50
Very Dense	Greater than 50



MODIFIED UNIFIED SOIL CLASSIFICATION

Major Divisions		Group Symbol	Typical Description
COARSE GRAINED SOIL	GRAVEL AND GRAVELLY SOILS	GW	Well-graded gravels or gravel-sand mixtures, little or no fines.
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines.
		GM	Silty gravels, gravel-sand-silt mixtures.
		GC	Clayey gravels, gravel-sand-clay mixtures.
	SAND AND SANDY SOILS	SW	Well-graded sands or gravelly sands, little or no fines.
		SP	Poorly-graded sands or gravelly sands, little or no fines.
		SM	Silty sands, sand-silt mixtures.
		SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS	SILT AND CLAY SOILS $W_L < 35\%$	ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
		OL	Organic silts and organic silty-clays of low plasticity.
	SILT AND CLAY SOILS $35\% < W_L < 50\%$	MI	Inorganic compressible fine sandy silt with clay of medium plasticity, clayey silts.
		CI	Inorganic clays of medium plasticity, silty clays.
		OI	Organic silty clays of medium plasticity.
	SILT AND CLAY SOILS $W_L > 50\%$	MH	Inorganic silts, micaceous or diatomaceous fine sandy of silty soils, elastic silts.
		CH	Inorganic clays of high plasticity, fat clays.
		OH	Organic clays of high plasticity, organic silts.
HIGHLY ORGANIC SOILS		Pt	Peat and other organic soils.

Note - W_L = Liquid Limit



EXPLANATION OF ROCK LOGGING TERMS

ROCK WEATHERING CLASSIFICATION

Fresh (FR)	No visible signs of weathering.
Fresh Jointed (FJ)	Weathering limited to surface of major discontinuities.
Slightly Weathered (SW)	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock materials.
Moderately Weathered (MW)	Weathering extends throughout the rock mass, but the rock material is not friable.
Highly Weathered (HW)	Weathering extends throughout the rock mass and the rock is partly friable.
Completely Weathered (CW)	Rock is wholly decomposed and in a friable condition, but the rock texture and structures are preserved.

TERMS

Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.
Solid Core Recovery: (SCR)	Percent ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.
Rock Quality Designation: (RQD)	Total length of sound core recovered in pieces 0.1 m in length or larger, as a percentage of total core length
Unconfined Compressive Strength: (UCS)	Axial stress required to break the specimen.
Fracture Index: (FI)	Frequency of natural fractures per 0.3 m of core run.

DISCONTINUITY SPACING

Bedding	Bedding Plane Spacing
Very thickly bedded	Greater than 2 m
Thickly bedded	0.6 to 2 m
Medium bedded	0.2 to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 to 60 mm
Laminated	6 to 20 mm
Thinly laminated	Less than 6 mm

STRENGTH CLASSIFICATION

Rock Strength	Approximate Uniaxial Compressive Strength (MPa)
Extremely Strong	Greater than 250
Very Strong	100 – 250
Strong	50 – 100
Medium Strong	25 – 50
Weak	5 – 25
Very Weak	1 – 5
Extremely Weak	0.25 – 1

RECORD OF BOREHOLE No 601

1 OF 1

METRIC

W.P. 4113-01-01 LOCATION 29-232/C Muskrat Creek Culvert, MTM Zone 9: N 5 049 722.1 E 278 947.1 ORIGINATED BY SMP
 HWY 17 BOREHOLE TYPE Portable BQ/NQ Casing COMPILED BY SMP
 DATUM Geodetic DATE 2015.10.19 - 2015.10.20 CHECKED BY KP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
148.8								20 40 60 80 100						
0.0	Gravel and sand with silt Loose to compact Brown FILL		1	SS	5									
			2	SS	15		148							
			3	SS	13		147							46 43 11 (SI+CL)
146.8														
2.1	SILTY SAND (SM) with gravel Compact Grey		4	SS	21									19 33 48 (SI+CL)
146.1														
2.7	BEDROCK Granite Slightly weathered Poor quality Grey		1	RUN			146							RUN #1 TCR=96% SCR=86% RQD=46%
145.4														
3.4	End of Borehole													

+³, ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 601A

1 OF 1

METRIC

W.P. 4113-01-01 LOCATION 29-232/C Muskrat Creek Culvert, MTM Zone 9: N 5 049 724.2 E 278 947.3 ORIGINATED BY SMP
 HWY 17 BOREHOLE TYPE Portable BQ/NQ Casing COMPILED BY SMP
 DATUM Geodetic DATE 2015.10.21 - 2015.10.21 CHECKED BY KP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					W P W W L				GR SA SI CL				
148.2	Advanced NQ Casing to 600 mm							20	40	60	80	100									
0.0	Gravel and sand with silt Loose to compact Brown FILL (Inferred)						148														
146.9							147														
1.2	BEDROCK Granite Slightly weathered Fair quality Grey		1	RUN																RUN #1 TCR=90% SCR=80% RQD=71%	
145.9							146														
2.3	End of Borehole																				

+³, ×³: Numbers refer to Sensitivity 20 15 10 5 0 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 602

1 OF 1

METRIC

W.P. 4113-01-01 LOCATION 29-232/C Muskrat Creek Culvert, MTM Zone 9: N 5 049 709.7 E 278 945.5 ORIGINATED BY CAM
 HWY 17 BOREHOLE TYPE HSA COMPILED BY SMP
 DATUM Geodetic DATE 2015.10.16 - 2015.10.16 CHECKED BY KP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								20 40 60 80 100						
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
							WATER CONTENT (%)							
							PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W _P W W _L							
149.6														
0.0	250 mm ASPHALT													
0.2	Sand with silt and gravel Compact Brown FILL						149							
	- Boulder at 1.2 m		1	SS	26									
			2	SS	14		148							39 52 9 (SI+CL)
147.3														
2.3	SAND (SP-SM) with silt Black to grey Very loose		3	SS	3		147							
			4	SS	3		146							0 90 10 (SI+CL)
			5	SS	2		145							
144.7			6	SS	2		144							
5.0	Clay (CL) with silt and gravel Firm Grey		7	SS	4		143							1 14 79 6
144.3			8	SS	3		142							
5.3	Silt (ML) with sand Very loose to loose Grey		9	SS	14		141							0 68 32 (SI+CL)
			10	SS	100/ 0mm									
142.8														
6.8	SILTY SAND (SM) TILL - frequent boulders Compact Grey													
	- Boulder at 7.6 m													
	- Boulder at 8.8 m													
140.5														
9.1	End of Borehole													

ONTMT4S 19-5161-263 MUSKRAT CREEK.GPJ 2012TEMPLATE(MTO).GDT 17/1/18

+³, ×³: Numbers refer to Sensitivity
 20
 15
 10
 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 603

1 OF 1

METRIC

W.P. 4113-01-01 LOCATION 29-232/C Muskrat Creek Culvert, MTM Zone 9: N 5 049 714.4 E 278 936.8 ORIGINATED BY SMP
 HWY 17 BOREHOLE TYPE HSA COMPILED BY SMP
 DATUM Geodetic DATE 2015.10.22 - 2015.10.22 CHECKED BY KP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					
								20 40 60 80 100					
								20 40 60 80 100					
149.6													
0.0	300 mm ASPHALT												
149.3													
0.3	Sand with silt and gravel Compact Brown FILL		1	SS	100/ 225mm		149						20 69 11 (SI+CL)
			2	SS	24		148						
			3	SS	18								
147.2													
2.4	SAND (SP-SM) with silt Very loose to loose Brown - trace organic matter		4	SS	6		147						2 89 9 (SI+CL)
			5	SS	6		146						
			6	SS	3		145						
			7	SS	4								
144.3													
5.3	SILT (ML) with sand Very loose to compact Grey		8	SS	1		144						0 20 74 6
			9	SS	WH		143						
			10	SS	10		142						
141.8			11	SS	100/ 150mm								
7.8	BEDROCK Granite Slightly weathered Fair quality Grey		1	RUN			141						RUN #1 TCR=80% SCR=67% RQD=51%
140.4													
9.2	End of Borehole												

ONTMT4S 19-5161-263 MUSKRAT CREEK GPJ 2012TEMPLATE(MTO).GDT 17/1/18

+³, ×³: Numbers refer to Sensitivity 20 15 10 5 0 5 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 604

1 OF 1

METRIC

W.P. 4113-01-01 LOCATION 29-232/C Muskrat Creek Culvert, MTM Zone 9: N 5 049 702.6 E 278 935.1 ORIGINATED BY SMP
 HWY 17 BOREHOLE TYPE Portable NQ Casing COMPILED BY SMP
 DATUM Geodetic DATE 2015.10.20 - 2015.10.21 CHECKED BY KP

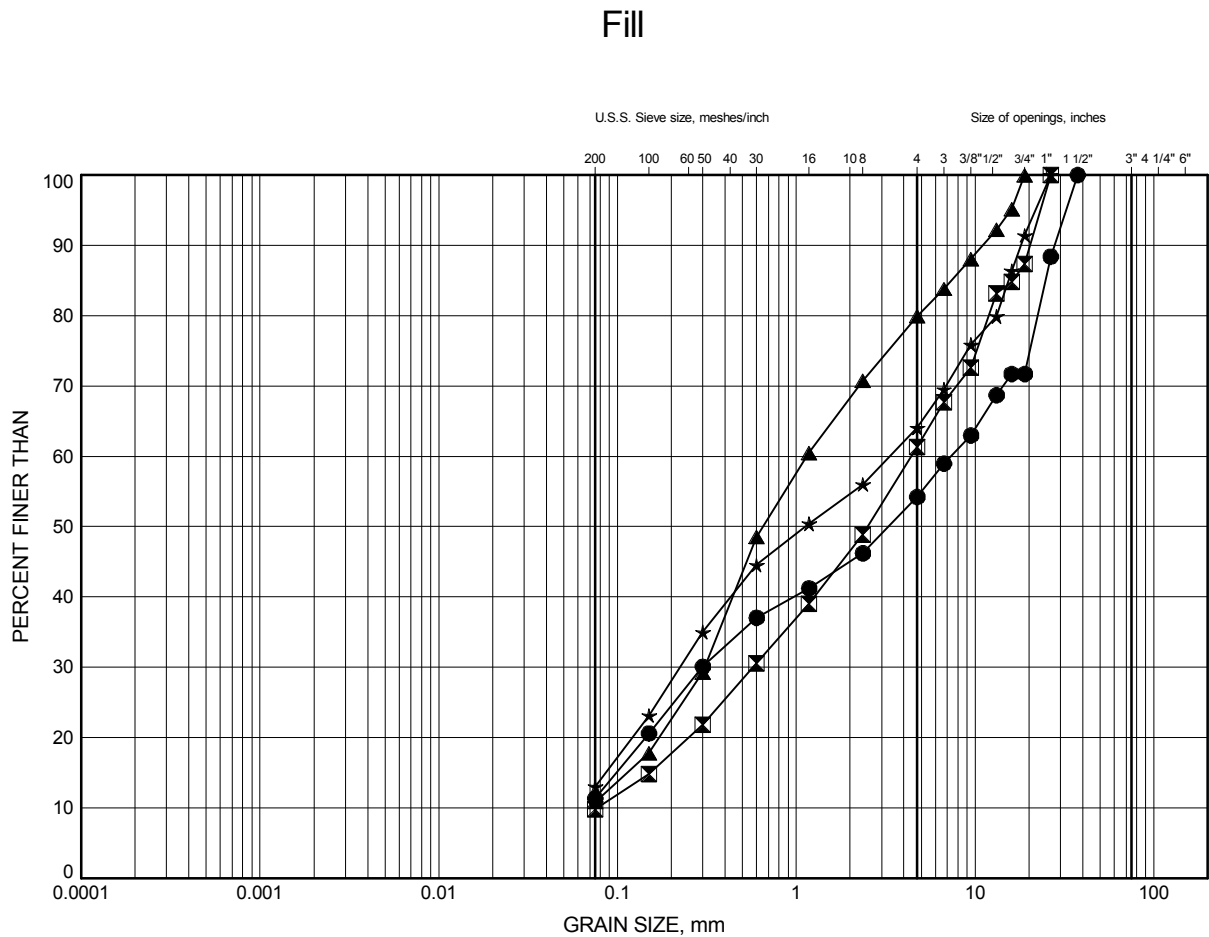
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										
148.6								20	40	60	80	100						
0.0	150 mm TOPSOIL							20	40	60	80	100						
0.2	Silty sand with gravel Loose to compact Brown FILL		1	SS	6		148											
			2	SS	19													
			3	SS	12		147											36 51 13 (SH+CL)
			4	SS	17													
146.2																		
2.4	SILT (ML) with sand Loose to compact Grey		5	SS	22		146											
			6	SS	15		145											
			7	SS	9													4 35 61 (SH+CL)
			8	SS	9		144											
			9	SS	8													
			10	SS	8		143											3 41 39 17
			11	SS	7		142											
141.9																		
6.7	BEDROCK Granite Slightly weathered Fair quality Grey		1	RUN													RUN #1 TCR=97% SCR=82% RQD=68%	
141.0																		
7.6	End of Borehole Groundwater level was measured in piezometer at 0.93 m BGS (elev. 147.7 m) on 2015/12/16																	

+³, ×³: Numbers refer to Sensitivity 20 15 10 5 0 5 10 (%) STRAIN AT FAILURE

APPENDIX C
LABORATORY TEST RESULTS

Site 29-323C - Muskrat Creek
GRAIN SIZE DISTRIBUTION

FIGURE 1



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	601	1.52	147.30
⊠	602	1.83	147.80
▲	603	1.07	148.54
★	604	1.52	147.08

Date November 2017
W.P. 4113-01-01



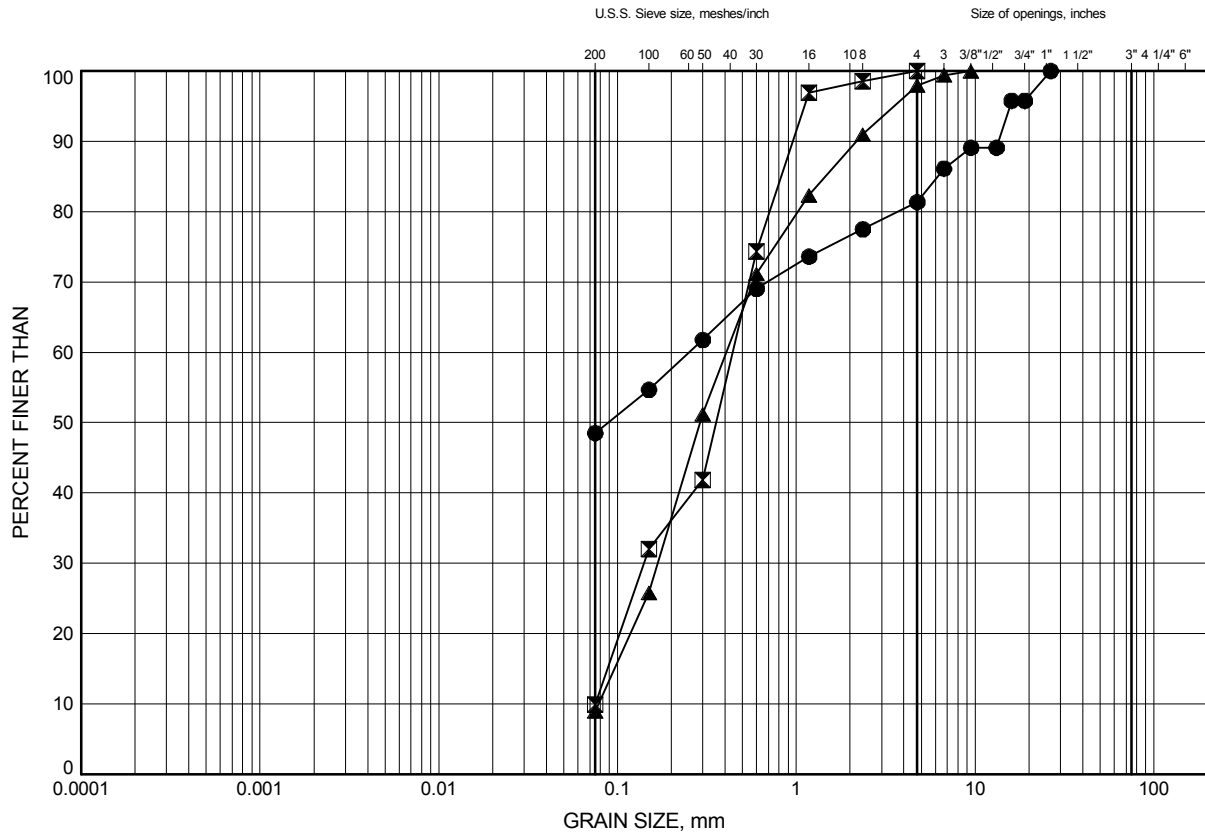
Prep'd KCP
Chkd. PC

Site 29-323C - Muskrat Creek

GRAIN SIZE DISTRIBUTION

FIGURE 2

Sand with Silt / Silty Sand



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	601	2.29	146.53
⊠	602	3.35	146.28
▲	603	3.17	146.44

Date November 2017
W.P. 4113-01-01

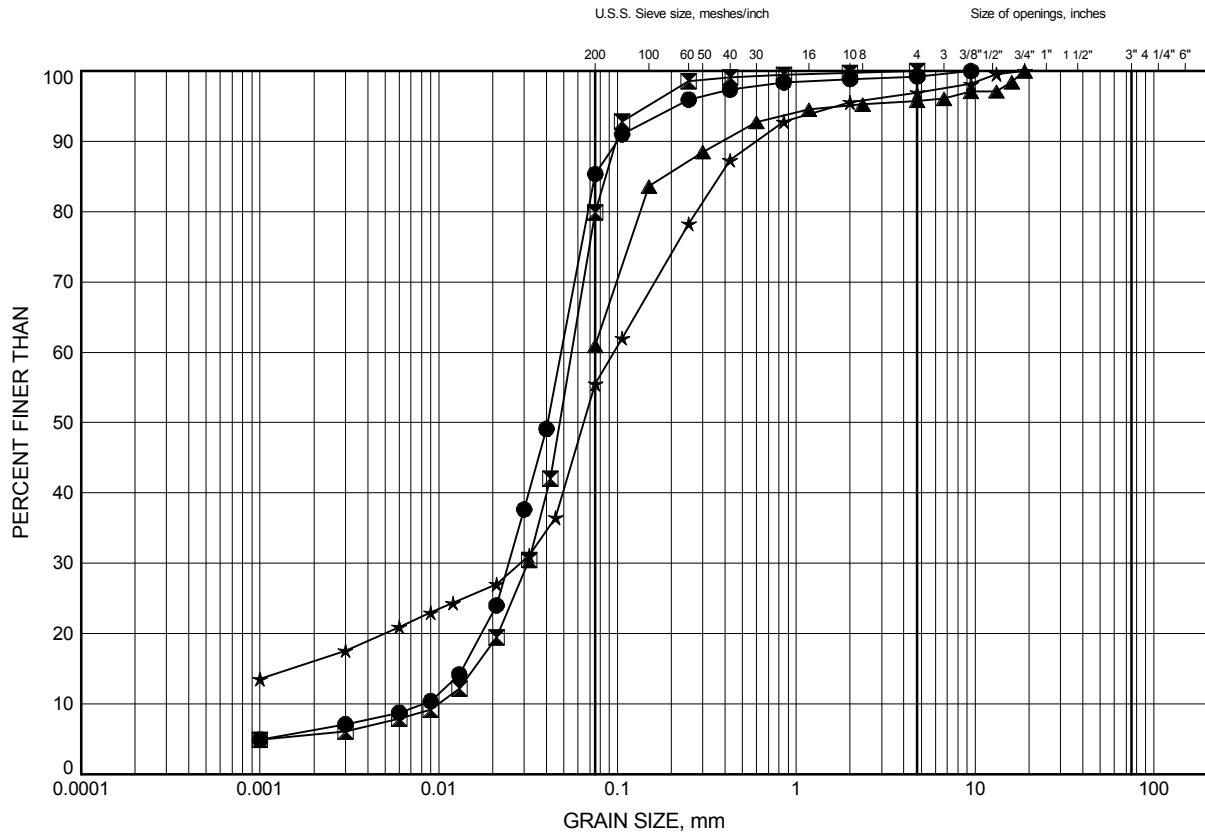


Prep'd KCP
Chkd. PC

Site 29-323C - Muskrat Creek
GRAIN SIZE DISTRIBUTION

FIGURE 3

Silt with Sand



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	602	6.40	143.23
⊠	603	5.64	143.97
▲	604	3.96	144.64
★	604	5.79	142.81

Date November 2017
W.P. 4113-01-01

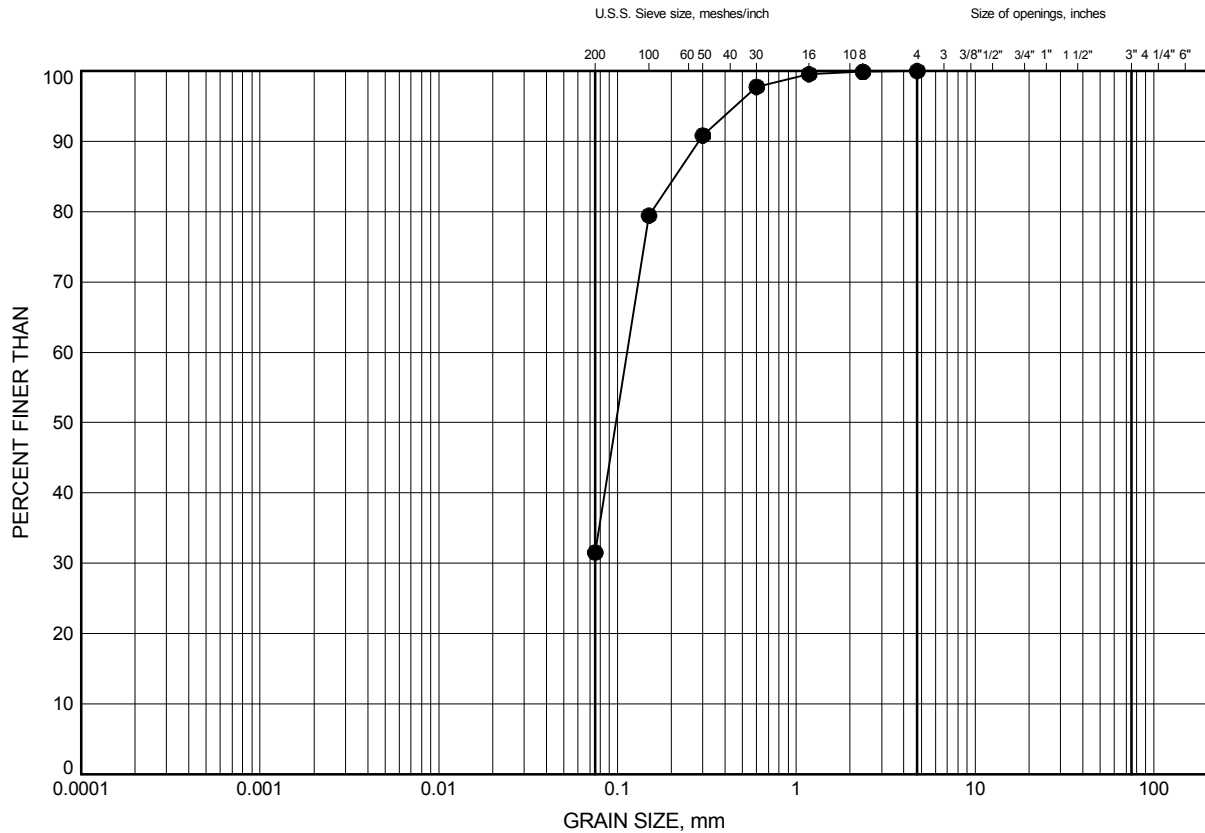


Prep'd KCP
Chkd. PC

Site 29-323C - Muskrat Creek
GRAIN SIZE DISTRIBUTION

FIGURE 4

Silty Sand: Till



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	602	7.09	142.54

Date January 2018
W.P. 4113-01-01



Prep'd KCP
Chkd. PC

Certificate of Analysis

Thurber Engineering Ltd.

2460 Lancaster Rd, Unit 107
Ottawa, ON K1B4S5
Attn: Kenton Power

Client PO:
Project: 19-5161-263
Custody:

Report Date: 13-Nov-2015
Order Date: 10-Nov-2015

Order #: 1546148

This Certificate of Analysis contains analytical data applicable to the following samples as submitted:

Paracel ID	Client ID
1546148-01	BH704 SS4 6' to 8'
1546148-02	BH601 SS4 6' to 8'
1546148-03	BH501 SS6 10.5' to 12.5'

Approved By:

Mark Foto

Mark Foto, M.Sc.
Lab Supervisor

Certificate of Analysis

Client: **Thurber Engineering Ltd.**

Client PO:

Report Date: 13-Nov-2015

Order Date: 10-Nov-2015

Project Description: **19-5161-263****Analysis Summary Table**

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC, water extraction	12-Nov-15	12-Nov-15
pH, soil	EPA 150.1 - pH probe @ 25 °C, CaCl buffered ext.	11-Nov-15	11-Nov-15
Resistivity	EPA 120.1 - probe, water extraction	12-Nov-15	12-Nov-15
Solids, %	Gravimetric, calculation	12-Nov-15	12-Nov-15

Certificate of Analysis
Client: Thurber Engineering Ltd.

Report Date: 13-Nov-2015

Order Date: 10-Nov-2015

Client PO:
Project Description: 19-5161-263

Client ID:		BH704 SS4 6' to 8'	BH601 SS4 6' to 8'	BH501 SS6 10.5' to 12.5'	-
Sample Date:		22-Oct-15	19-Oct-15	27-Oct-15	-
Sample ID:		1546148-01	1546148-02	1546148-03	-
MDL/Units		Soil	Soil	Soil	-
Physical Characteristics					
% Solids	0.1 % by Wt.	81.9	76.3	91.8	-
General Inorganics					
pH	0.05 pH Units	7.56	7.73	7.99	-
Resistivity	0.10 Ohm.m	25.3	31.2	157	-
Anions					
Chloride	5 ug/g dry	129	70	6	-
Sulphate	5 ug/g dry	27	112	7	-

Certificate of Analysis

Client: **Thurber Engineering Ltd.**

Client PO:

Report Date: 13-Nov-2015

Order Date: 10-Nov-2015

Project Description: **19-5161-263**

Method Quality Control: Blank

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	ND	5	ug/g						
Sulphate	ND	5	ug/g						
General Inorganics									
Resistivity	ND	0.10	Ohm.m						

Certificate of Analysis

Client: **Thurber Engineering Ltd.**

Client PO:

Report Date: 13-Nov-2015

Order Date: 10-Nov-2015

Project Description: **19-5161-263**

Method Quality Control: Duplicate

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	7.0	5	ug/g dry	7.1			0.5	20	
Sulphate	24.3	5	ug/g dry	25.1			3.6	20	
General Inorganics									
pH	8.11	0.05	pH Units	7.99			1.5	10	
Resistivity	24.8	0.10	Ohm.m	25.3			1.9	20	
Physical Characteristics									
% Solids	78.2	0.1	% by Wt.	77.6			0.7	25	

Certificate of Analysis

Client: **Thurber Engineering Ltd.**

Client PO:

Report Date: 13-Nov-2015

Order Date: 10-Nov-2015

Project Description: **19-5161-263****Method Quality Control: Spike**

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	100	5	ug/g	7.1	93.3	78-113			
Sulphate	104	5	ug/g	25.1	79.1	78-111			

Certificate of Analysis

Client: **Thurber Engineering Ltd.**

Client PO:

Report Date: 13-Nov-2015

Order Date: 10-Nov-2015

Project Description: **19-5161-263**

Qualifier Notes:

None

Sample Data Revisions

None

Work Order Revisions / Comments:

None

Other Report Notes:

n/a: not applicable

ND: Not Detected

MDL: Method Detection Limit

Source Result: Data used as source for matrix and duplicate samples

%REC: Percent recovery.

RPD: Relative percent difference.

Soil results are reported on a dry weight basis when the units are denoted with 'dry'.

Where %Solids is reported, moisture loss includes the loss of volatile hydrocarbons.

APPENDIX D
SEISMIC CONE PENETRATION TEST RESULTS

PRESENTATION OF SITE INVESTIGATION RESULTS

Highway 17 – Muskrat Creek Culvert

Prepared for:

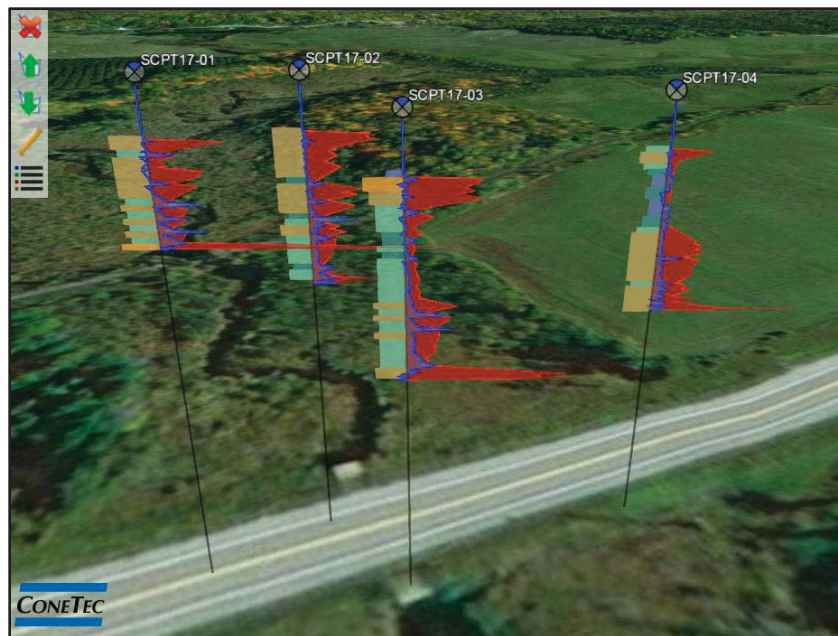
Thurber Engineering Ltd.

ConeTec Job No: 17-05021

Project Start Date: 23-May-2017

Project End Date: 23-May-2017

Report Date: 25-May-2017



Prepared by:

ConeTec Investigations Ltd.
9033 Leslie Street, Unit 15
Richmond Hill, ON L4B 4K3

Tel: (905) 886-2663

Fax: (905) 886-2664

Toll Free: (800) 504-1116

Email: conetecON@conetec.com

www.conetec.com

www.conetecdataservices.com



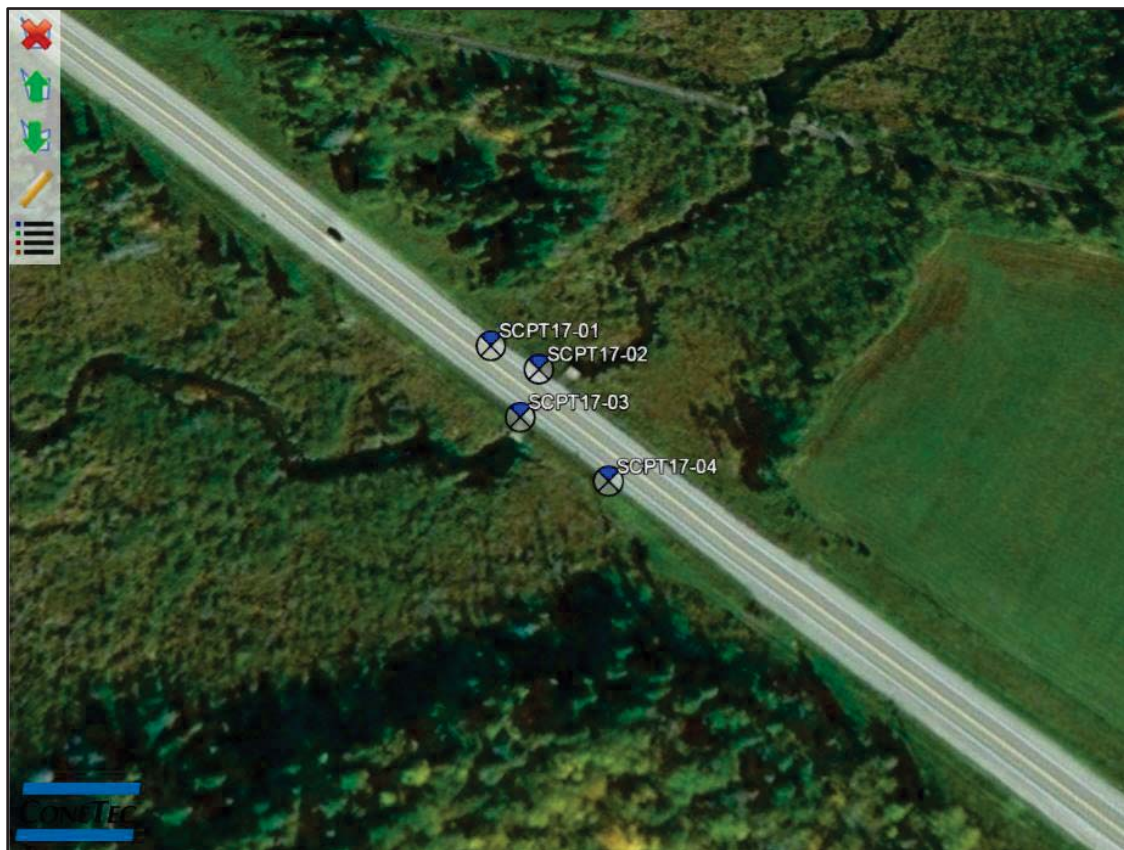
Introduction

The enclosed report presents the results of the site investigation program conducted by ConeTec Investigations Ltd. for Thurber Engineering Ltd. on Highway 17 at the Muskrat Creek Culvert. The program consisted of four seismic cone penetration tests (SCPT) performed on May 23, 2017.

Project Information

Project	
Client	Thurber Engineering Ltd.
Project	Highway 17 - Muskrat Creek Culvert
ConeTec project number	17-05021

A map from Google Earth including the SCPT test locations is presented below.



Rig Description	Deployment System	Test Type
CPT truck rig (C3)	30 ton rig cylinder	SCPT

Coordinates		
Test Type	Collection Method	EPSG Number
SCPT	Consumer grade GPS	32618

Cone Penetration Test (CPT)	
Depth reference	Depths are referenced to the existing ground surface at the time of each test.
Tip and sleeve data offset	0.1 meter This has been accounted for in the CPT data files.
Additional plots	Advanced CPT plots displaying I_c , $S_u(Nkt)$, and $N1(60) I_c$, along with seismic CPT plots are provided.

Cone Penetrometers Used for this Project						
Cone Description	Cone Number	Cross Sectional Area (cm ²)	Sleeve Area (cm ²)	Tip Capacity (bar)	Sleeve Capacity (bar)	Pore Pressure Capacity (psi)
379:T1500F15U500	379	15	225	1500	15	500
Cone 379 was used for both CPT soundings.						

Interpretation Tables	
Additional information	<p>The Normalized Qtn Soil Behaviour Type (SBT-Qtn) classification chart (Robertson, 2009) was used to classify the soil for this project. A detailed set of CPT interpretations were generated and are provided in Excel format files in the release folder. The CPT interpretations are based on values of corrected tip (q_t), sleeve friction (f_s) and pore pressure (u_2).</p> <p>Soils were classified as either drained or undrained based on the Normalized Qtn Soil Behaviour Type (SBT-Qtn) classification chart (Robertson, 2009). Calculations for both drained and undrained parameters were included for materials that classified as silt mixtures – clayey silt to silty clay (zone 4).</p>

Limitations

This report has been prepared for the exclusive use of Thurber Engineering Ltd. (Client) for the project titled “Highway 17 - Muskrat Creek Culvert”. The report’s contents may not be relied upon by any other party without the express written permission of ConeTec Investigations Ltd. (ConeTec). ConeTec has provided site investigation services, prepared the factual data reporting, and provided geotechnical parameter calculations consistent with current best practices. No other warranty, expressed or implied, is made.

The information presented in the report document and the accompanying data set pertain to the specific project, site conditions and objectives described to ConeTec by the Client. In order to properly understand the factual data, assumptions and calculations, reference must be made to the documents provided and their accompanying data sets, in their entirety.

The cone penetration tests (CPTu) are conducted using an integrated electronic piezocone penetrometer and data acquisition system manufactured by Adara Systems Ltd. of Richmond, British Columbia, Canada.

ConeTec's piezocone penetrometers are compression type designs in which the tip and friction sleeve load cells are independent and have separate load capacities. The piezocones use strain gauged load cells for tip and sleeve friction and a strain gauged diaphragm type transducer for recording pore pressure. The piezocones also have a platinum resistive temperature device (RTD) for monitoring the temperature of the sensors, an accelerometer type dual axis inclinometer and a geophone sensor for recording seismic signals. All signals are amplified down hole within the cone body and the analog signals are sent to the surface through a shielded cable.

ConeTec penetrometers are manufactured with various tip, friction and pore pressure capacities in both 10 cm² and 15 cm² tip base area configurations in order to maximize signal resolution for various soil conditions. The specific piezocone used for each test is described in the CPT summary table presented in the first Appendix. The 15 cm² penetrometers do not require friction reducers as they have a diameter larger than the deployment rods. The 10 cm² piezocones use a friction reducer consisting of a rod adapter extension behind the main cone body with an enlarged cross sectional area (typically 44 mm diameter over a length of 32 mm with tapered leading and trailing edges) located at a distance of 585 mm above the cone tip.

The penetrometers are designed with equal end area friction sleeves, a net end area ratio of 0.8 and cone tips with a 60 degree apex angle.

All ConeTec piezocones can record pore pressure at various locations. Unless otherwise noted, the pore pressure filter is located directly behind the cone tip in the "u₂" position (ASTM Type 2). The filter is 6 mm thick, made of porous plastic (polyethylene) having an average pore size of 125 microns (90-160 microns). The function of the filter is to allow rapid movements of extremely small volumes of water needed to activate the pressure transducer while preventing soil ingress or blockage.

The piezocone penetrometers are manufactured with dimensions, tolerances and sensor characteristics that are in general accordance with the current ASTM D5778 standard. ConeTec's calibration criteria also meets or exceeds those of the current ASTM D5778 standard. An illustration of the piezocone penetrometer is presented in Figure CPTu.

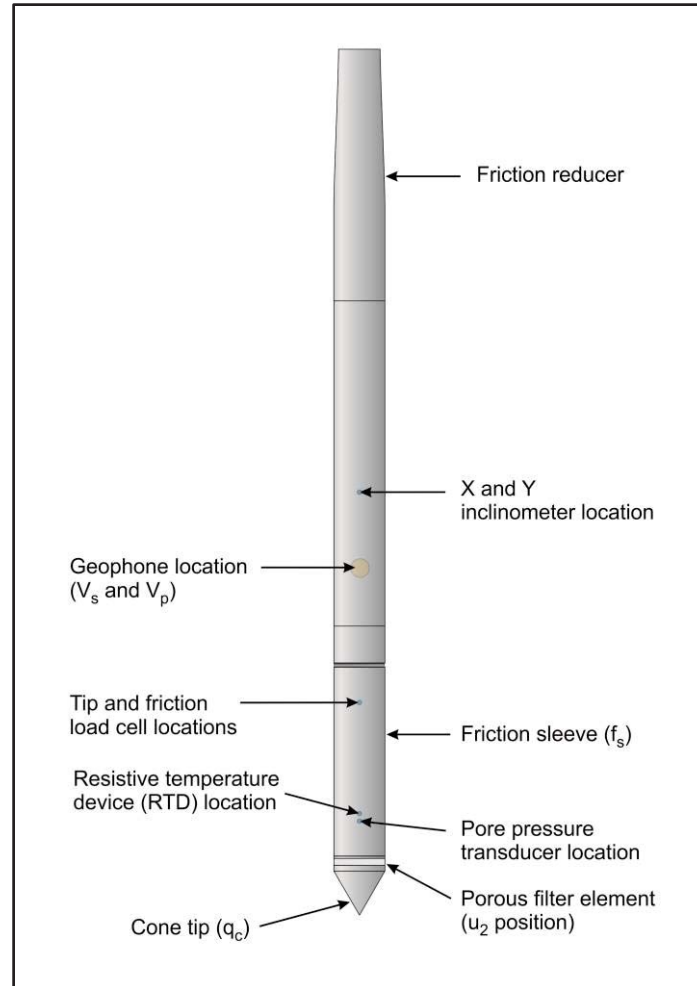


Figure CPTu. Piezocone Penetrometer (15 cm²)

The ConeTec data acquisition systems consist of a Windows based computer and a signal conditioner and power supply interface box with a 16 bit (or greater) analog to digital (A/D) converter. The data is recorded at fixed depth increments using a depth wheel attached to the push cylinders or by using a spring loaded rubber depth wheel that is held against the cone rods. The typical recording intervals are either 2.5 cm or 5.0 cm depending on project requirements; custom recording intervals are possible. The system displays the CPTu data in real time and records the following parameters to a storage media during penetration:

- Depth
- Uncorrected tip resistance (q_c)
- Sleeve friction (f_s)
- Dynamic pore pressure (u)
- Additional sensors such as resistivity, passive gamma, ultra violet induced fluorescence, if applicable

All testing is performed in accordance to ConeTec's CPT operating procedures which are in general accordance with the current ASTM D5778 standard.

Prior to the start of a CPTu sounding a suitable cone is selected, the cone and data acquisition system are powered on, the pore pressure system is saturated with either glycerine or silicone oil and the baseline readings are recorded with the cone hanging freely in a vertical position.

The CPTu is conducted at a steady rate of 2 cm/s, within acceptable tolerances. Typically one meter length rods with an outer diameter of 1.5 inches are added to advance the cone to the sounding termination depth. After cone retraction final baselines are recorded.

Additional information pertaining to ConeTec's cone penetration testing procedures:

- Each filter is saturated in silicone oil or glycerine under vacuum pressure prior to use
- Recorded baselines are checked with an independent multi-meter
- Baseline readings are compared to previous readings
- Soundings are terminated at the client's target depth or at a depth where an obstruction is encountered, excessive rod flex occurs, excessive inclination occurs, equipment damage is likely to take place, or a dangerous working environment arises
- Differences between initial and final baselines are calculated to ensure zero load offsets have not occurred and to ensure compliance with ASTM standards

The interpretation of piezocone data for this report is based on the corrected tip resistance (q_t), sleeve friction (f_s) and pore water pressure (u). The interpretation of soil type is based on the correlations developed by Robertson (1990) and Robertson (2009). It should be noted that it is not always possible to accurately identify a soil type based on these parameters. In these situations, experience, judgment and an assessment of other parameters may be used to infer soil behaviour type.

The recorded tip resistance (q_c) is the total force acting on the piezocone tip divided by its base area. The tip resistance is corrected for pore pressure effects and termed corrected tip resistance (q_t) according to the following expression presented in Robertson et al, 1986:

$$q_t = q_c + (1-a) \cdot u_2$$

where: q_t is the corrected tip resistance

q_c is the recorded tip resistance

u_2 is the recorded dynamic pore pressure behind the tip (u_2 position)

a is the Net Area Ratio for the piezocone (0.8 for ConeTec probes)

The sleeve friction (f_s) is the frictional force on the sleeve divided by its surface area. As all ConeTec piezocones have equal end area friction sleeves, pore pressure corrections to the sleeve data are not required.

The dynamic pore pressure (u) is a measure of the pore pressures generated during cone penetration. To record equilibrium pore pressure, the penetration must be stopped to allow the dynamic pore pressures to stabilize. The rate at which this occurs is predominantly a function of the permeability of the soil and the diameter of the cone.

The friction ratio (R_f) is a calculated parameter. It is defined as the ratio of sleeve friction to the tip resistance expressed as a percentage. Generally, saturated cohesive soils have low tip resistance, high

friction ratios and generate large excess pore water pressures. Cohesionless soils have higher tip resistances, lower friction ratios and do not generate significant excess pore water pressure.

A summary of the CPTu soundings along with test details and individual plots are provided in the appendices. A set of interpretation files were generated for each sounding based on published correlations and are provided in Excel format in the data release folder. Information regarding the interpretation methods used is also included in the data release folder.

For additional information on CPTu interpretations, refer to Robertson et al. (1986), Lunne et al. (1997), Robertson (2009), Mayne (2013, 2014) and Mayne and Peuchen (2012).

Shear wave velocity testing is performed in conjunction with the piezocone penetration test (SCPTu) in order to collect interval velocities. For some projects seismic compression wave (V_p) velocity is also determined.

ConeTec's piezocone penetrometers are manufactured with a horizontally active geophone (28 hertz) that is rigidly mounted in the body of the cone penetrometer, 0.2 meters behind the cone tip.

Shear waves are typically generated by using an impact hammer horizontally striking a beam that is held in place by a normal load. In some instances an auger source or an imbedded impulsive source maybe used for both shear waves and compression waves. The hammer and beam act as a contact trigger that triggers the recording of the seismic wave traces. For impulsive devices an accelerometer trigger may be used. The traces are recorded using an up-hole integrated digital oscilloscope which is part of the SCPTu data acquisition system. An illustration of the shear wave testing configuration is presented in Figure SCPTu-1.

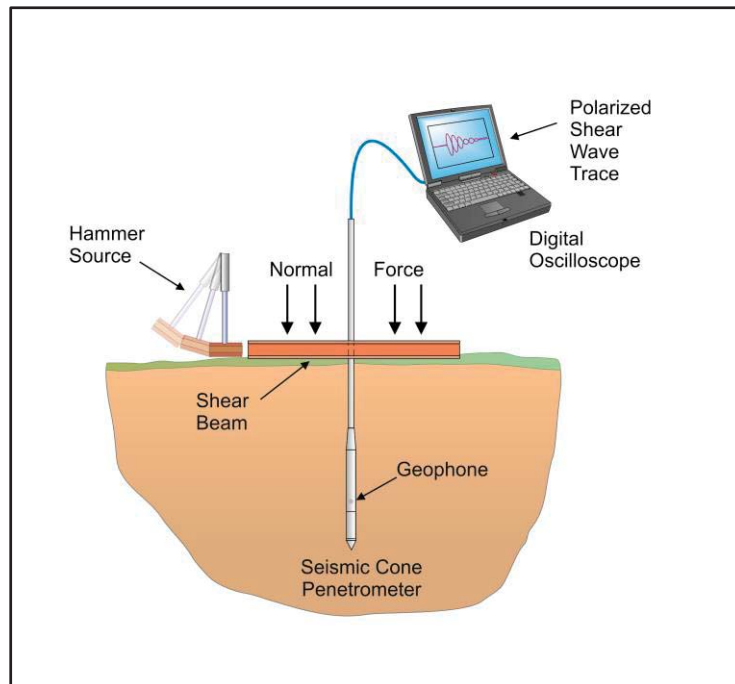


Figure SCPTu-1. Illustration of the SCPTu system

All testing is performed in accordance to ConeTec's SCPTu operating procedures.

Prior to the start of a SCPTu sounding, the procedures described in the Cone Penetration Test section are followed. In addition, the active axis of the geophone is aligned parallel to the beam (or source) and the horizontal offset between the cone and the source is measured and recorded.

Prior to recording seismic waves at each test depth, cone penetration is stopped and the rods are decoupled from the rig to avoid transmission of rig energy down the rods. Multiple wave traces are recorded for quality control purposes. After reviewing wave traces for consistency the cone is pushed to the next test depth (typically one meter intervals or as requested by the client). Figure SCPTu-2 presents an illustration of a SCPTu test.

For additional information on seismic cone penetration testing refer to Robertson et.al. (1986).

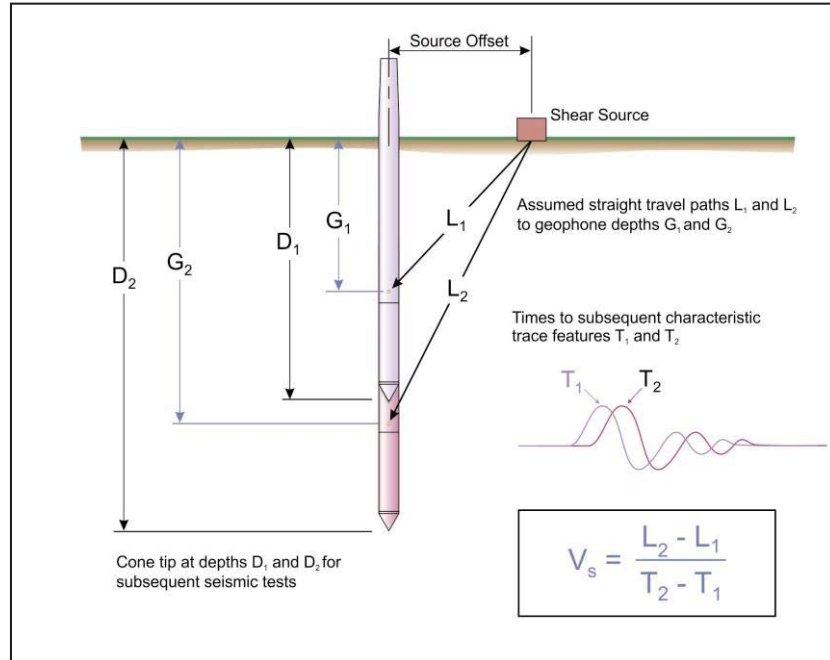


Figure SCPTu-2. Illustration of a seismic cone penetration test

Calculation of the interval velocities are performed by visually picking a common feature (e.g. the first characteristic peak, trough, or crossover) on all of the recorded wave sets and taking the difference in ray path divided by the time difference between subsequent features. Ray path is defined as the straight line distance from the seismic source to the geophone, accounting for beam offset, source depth and geophone offset from the cone tip.

The average shear wave velocity to a depth of 30 meters (V_{s30}) has been calculated and provided for all applicable soundings using an equation presented in Crow et al., 2012.

$$V_{s30} = \frac{\text{total thickness of all layers (30m)}}{\sum(\text{layer traveltimes})}$$

The layer travel times refers to the travel times propagating in the vertical direction, not the measured travel times from an offset source.

Tabular results and SCPTu plots are presented in the relevant appendix.

The cone penetration test is halted at specific depths to carry out pore pressure dissipation (PPD) tests, shown in Figure PPD-1. For each dissipation test the cone and rods are decoupled from the rig and the data acquisition system measures and records the variation of the pore pressure (u) with time (t).

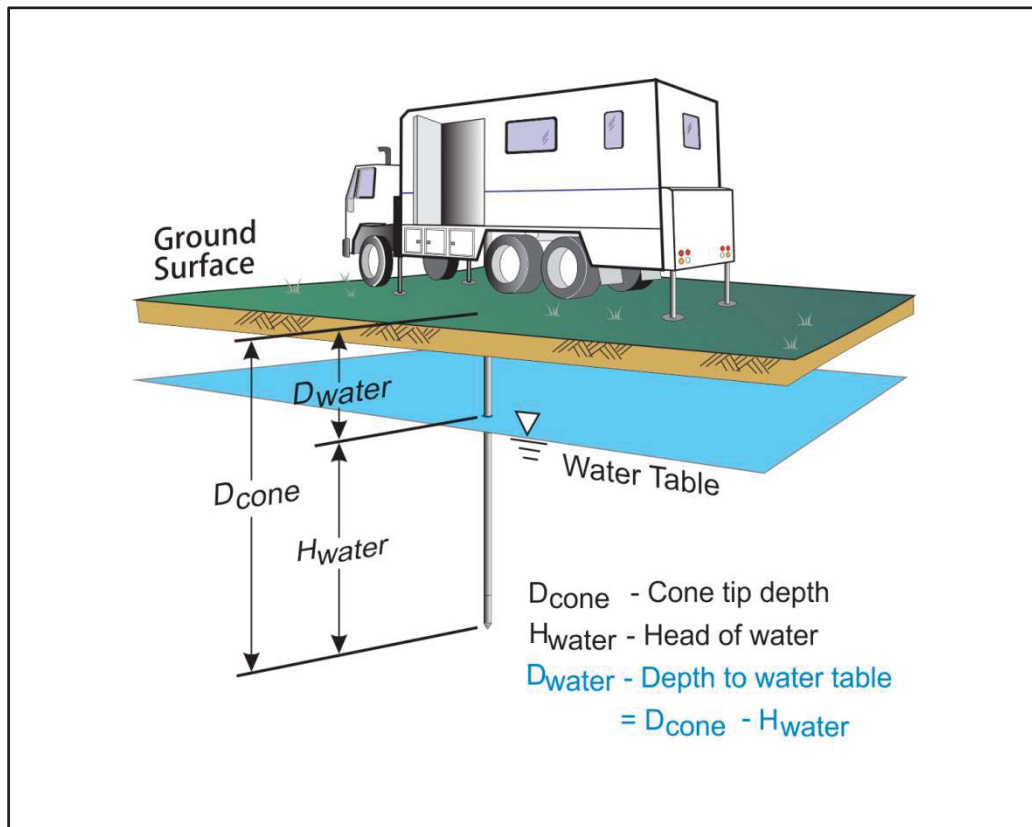


Figure PPD-1. Pore pressure dissipation test setup

Pore pressure dissipation data can be interpreted to provide estimates of ground water conditions, permeability, consolidation characteristics and soil behaviour.

The typical shapes of dissipation curves shown in Figure PPD-2 are very useful in assessing soil type, drainage, in situ pore pressure and soil properties. A flat curve that stabilizes quickly is typical of a freely draining sand. Undrained soils such as clays will typically show positive excess pore pressure and have long dissipation times. Dilative soils will often exhibit dynamic pore pressures below equilibrium that then rise over time. Overconsolidated fine-grained soils will often exhibit an initial dilatory response where there is an initial rise in pore pressure before reaching a peak and dissipating.

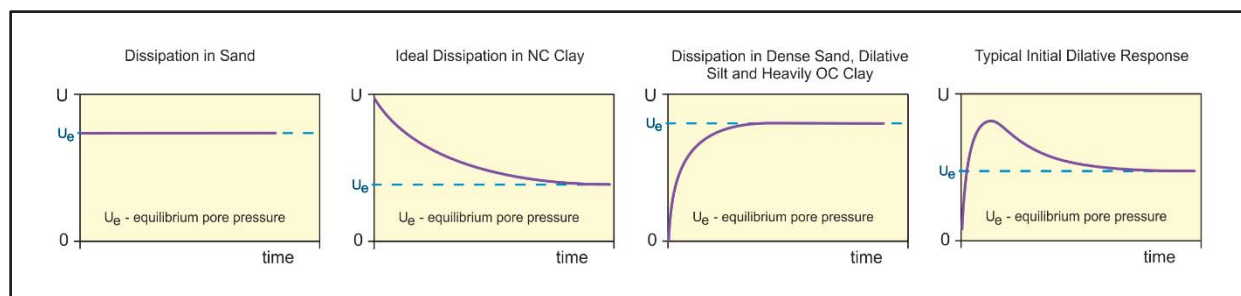


Figure PPD-2. Pore pressure dissipation curve examples

In order to interpret the equilibrium pore pressure (u_{eq}) and the apparent phreatic surface, the pore pressure should be monitored until such time as there is no variation in pore pressure with time as shown for each curve of Figure PPD-2.

In fine grained deposits the point at which 100% of the excess pore pressure has dissipated is known as t_{100} . In some cases this can take an excessive amount of time and it may be impractical to take the dissipation to t_{100} . A theoretical analysis of pore pressure dissipations by Teh and Houlsby (1991) showed that a single curve relating degree of dissipation versus theoretical time factor (T^*) may be used to calculate the coefficient of consolidation (c_h) at various degrees of dissipation resulting in the expression for c_h shown below.

$$c_h = \frac{T^* \cdot a^2 \cdot \sqrt{I_r}}{t}$$

Where:

- T^* is the dimensionless time factor (Table Time Factor)
- a is the radius of the cone
- I_r is the rigidity index
- t is the time at the degree of consolidation

Table Time Factor. T^* versus degree of dissipation (Teh and Houlsby, 1991)

Degree of Dissipation (%)	20	30	40	50	60	70	80
$T^* (u_2)$	0.038	0.078	0.142	0.245	0.439	0.804	1.60

The coefficient of consolidation is typically analyzed using the time (t_{50}) corresponding to a degree of dissipation of 50% (u_{50}). In order to determine t_{50} , dissipation tests must be taken to a pressure less than u_{50} . The u_{50} value is half way between the initial maximum pore pressure and the equilibrium pore pressure value, known as u_{100} . To estimate u_{50} , both the initial maximum pore pressure and u_{100} must be known or estimated. Other degrees of dissipations may be considered, particularly for extremely long dissipations.

At any specific degree of dissipation the equilibrium pore pressure (u at t_{100}) must be estimated at the depth of interest. The equilibrium value may be determined from one or more sources such as measuring the value directly (u_{100}), estimating it from other dissipations in the same profile, estimating the phreatic surface and assuming hydrostatic conditions, from nearby soundings, from client provided information, from site observations and/or past experience, or from other site instrumentation.

For calculations of c_h (Teh and Houlsby, 1991), t_{50} values are estimated from the corresponding pore pressure dissipation curve and a rigidity index (I_r) is assumed. For curves having an initial dilatory response in which an initial rise in pore pressure occurs before reaching a peak, the relative time from the peak value is used in determining t_{50} . In cases where the time to peak is excessive, t_{50} values are not calculated.

Due to possible inherent uncertainties in estimating I_r , the equilibrium pore pressure and the effect of an initial dilatory response on calculating t_{50} , other methods should be applied to confirm the results for c_h .

Additional published methods for estimating the coefficient of consolidation from a piezocone test are described in Burns and Mayne (1998, 2002), Jones and Van Zyl (1981), Robertson et al. (1992) and Sully et al. (1999).

A summary of the pore pressure dissipation tests and dissipation plots are presented in the relevant appendix.

REFERENCES

- ASTM D5778-12, 2012, "Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils", ASTM, West Conshohocken, US.
- Burns, S.E. and Mayne, P.W., 1998, "Monotonic and dilatatory pore pressure decay during piezocone tests", *Canadian Geotechnical Journal* 26 (4): 1063-1073.
- Burns, S.E. and Mayne, P.W., 2002, "Analytical cavity expansion-critical state model cone dissipation in fine-grained soils", *Soils & Foundations*, Vol. 42(2): 131-137.
- Crow, H.L., Hunter, J.A., Bobrowsky, P.T., 2012, "National shear wave measurement guidelines for Canadian seismic site assessment", *GeoManitoba 2012*, Sept 30 to Oct 2, Winnipeg, Manitoba.
- Jones, G.A. and Van Zyl, D.J.A., 1981, "The piezometer probe: a useful investigation tool", *Proceedings, 10th International Conference on Soil Mechanics and Foundation Engineering*, Vol. 3, Stockholm: 489-495.
- Lunne, T., Robertson, P.K. and Powell, J. J. M., 1997, "Cone Penetration Testing in Geotechnical Practice", Blackie Academic and Professional.
- Mayne, P.W., 2013, "Evaluating yield stress of soils from laboratory consolidation and in-situ cone penetration tests", *Sound Geotechnical Research to Practice (Holtz Volume) GSP 230*, ASCE, Reston/VA: 406-420.
- Mayne, P.W., 2014, "Interpretation of geotechnical parameters from seismic piezocone tests", *CPT'14 Keynote Address*, Las Vegas, NV, May 2014.
- Mayne, P.W. and Peuchen, J., 2012, "Unit weight trends with cone resistance in soft to firm clays", *Geotechnical and Geophysical Site Characterization 4*, Vol. 1 (Proc. ISC-4, Pernambuco), CRC Press, London: 903-910.
- Robertson, P.K., 1990, "Soil Classification Using the Cone Penetration Test", *Canadian Geotechnical Journal*, Volume 27: 151-158.
- Robertson, P.K., 2009, "Interpretation of cone penetration tests – a unified approach", *Canadian Geotechnical Journal*, Volume 46: 1337-1355.
- Robertson, P.K., Campanella, R.G., Gillespie, D. and Greig, J., 1986, "Use of Piezometer Cone Data", *Proceedings of InSitu 86*, ASCE Specialty Conference, Blacksburg, Virginia.
- Robertson, P.K., Campanella, R.G., Gillespie D and Rice, A., 1986, "Seismic CPT to Measure In-Situ Shear Wave Velocity", *Journal of Geotechnical Engineering ASCE*, Vol. 112, No. 8: 791-803.
- Robertson, P.K., Sully, J.P., Woeller, D.J., Lunne, T., Powell, J.J.M. and Gillespie, D.G., 1992, "Estimating coefficient of consolidation from piezocone tests", *Canadian Geotechnical Journal*, 29(4): 551-557.
- Sully, J.P., Robertson, P.K., Campanella, R.G. and Woeller, D.J., 1999, "An approach to evaluation of field CPTU dissipation data in overconsolidated fine-grained soils", *Canadian Geotechnical Journal*, 36(2): 369-381.

REFERENCES

Teh, C.I., and Houlsby, G.T., 1991, "An analytical study of the cone penetration test in clay", *Geotechnique*, 41(1): 17-34.

The appendices listed below are included in the report:

- Cone Penetration Test Summary and Standard Cone Penetration Test Plots
- Advanced Cone Penetration Test Plots displaying I_c , $S_u(Nkt)$, and $N1(60)I_c$
- Seismic Cone Penetration Test Plots
- Seismic Cone Penetration Test Tabular Results
- Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

Cone Penetration Test Summary and Standard Cone Penetration Test Plots

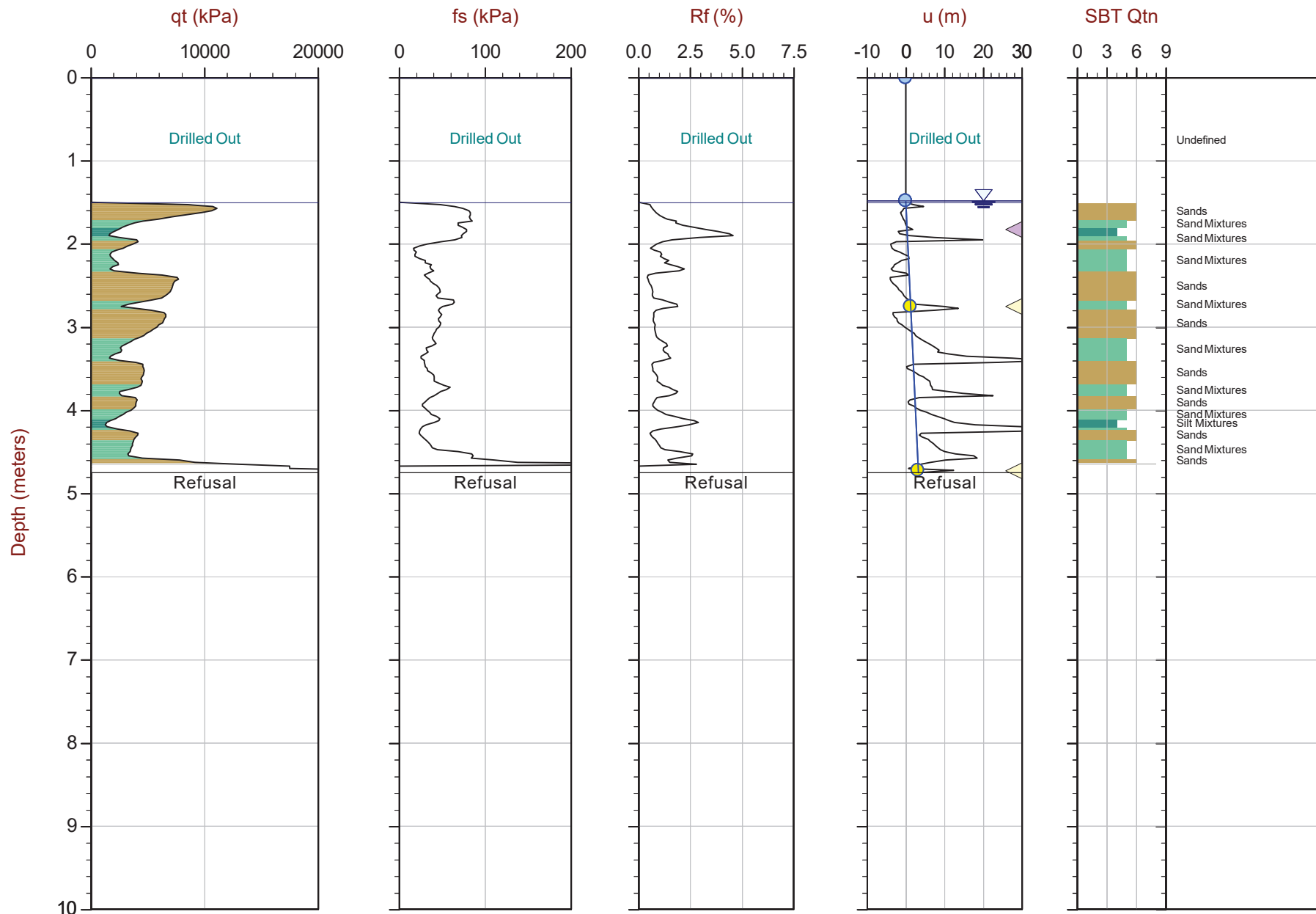


Job No: 17-05021
Client: Thurber Engineering
Project: Hwy 17 - Muskrat Creek Culvert
Start Date: 23-May-2017
End Date: 23-May-2017

CONE PENETRATION TEST SUMMARY

Sounding ID	File Name	Date	Cone	Assumed Phreatic Surface ¹ (m)	Final Depth (m)	Northing ² (m)	Easting (m)	Elevation ³ (m)	Refer to Notation Number
SCPT17-01	17-05021_SP01	23-May-2017	379:T1500F15U500	1.5	4.750	5049789	357129	149.6	
SCPT17-02	17-05021_SP02	23-May-2017	379:T1500F15U500	1.7	6.700	5049779	357140	149.6	
SCPT17-03	17-05021_SP03	23-May-2017	379:T1500F15U500	1.7	8.025	5049767	357132	149.6	
SCPT17-04	17-05021_SP04	23-May-2017	379:T1500F15U500	1.5	7.325	5049744	357150	149.7	






1. The assumed phreatic surface was based on pore pressure dissipation tests. Hydrostatic conditions were assumed for the calculated parameters.
2. Coordinates were collected with a consumer grade GPS device in datum WGS84/UTM Zone 18 North.
3. Elevations were provided by the client.



Max Depth: 4.750 m / 15.58 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 17-05021_SP01.COR
UnitWt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 18N: 5049789m E: 357129m Elev: 149.6m
Sheet No: 1 of 1

 Equilibrium Pore Pressure (Ueq)
  Assumed Ueq
  Dissipation, Ueq achieved
  Dissipation, Ueq not achieved
  Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



Thurber Engineering

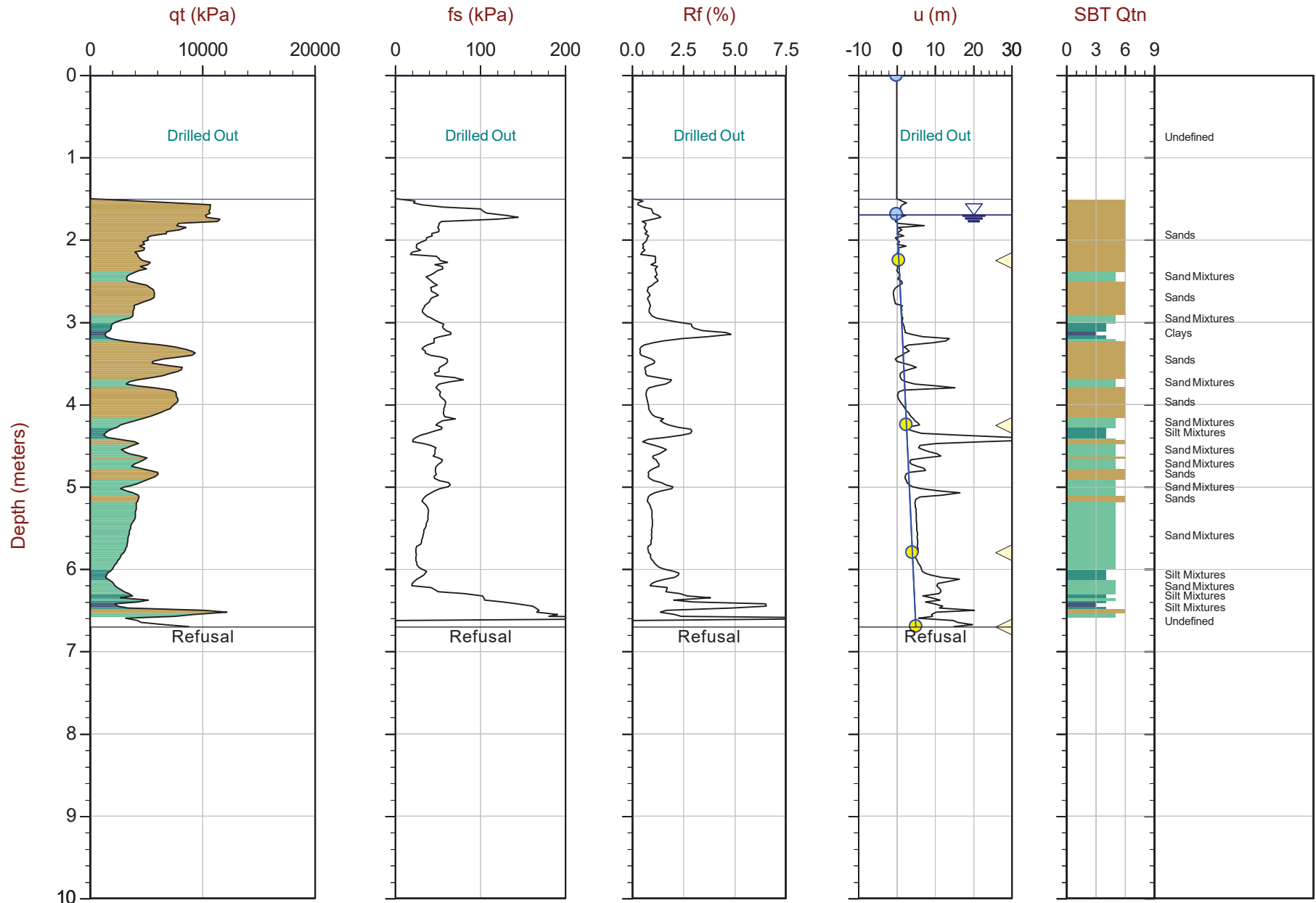
Job No: 17-05021

Date: 2017-05-23 10:19

Site: Hwy 17 - Muskrat Creek Culvert

Sounding: SCPT17-02

Cone: 379:T1500F15U500



Max Depth: 6.700 m / 21.98 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: Every Point

File: 17-05021_SP02.COR

Unit Wt: SBTQtn(PKR2009)

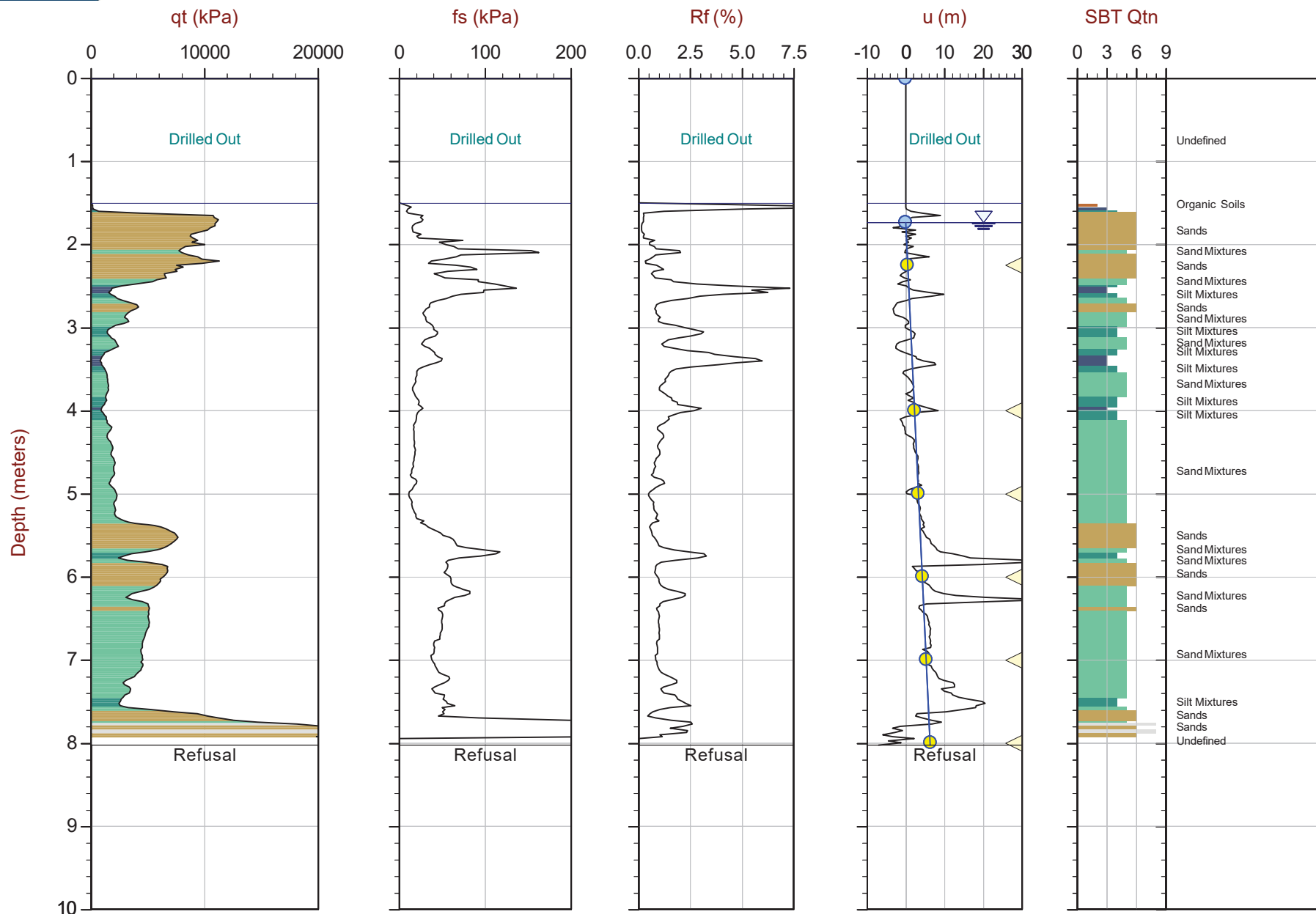
SBT: Robertson, 2009 and 2010

Coords: UTM 18N N: 5049779m E: 357140m Elev: 149.6m

Sheet No: 1 of 1

● Equilibrium Pore Pressure (Ueq) ● Assumed Ueq ▲ Dissipation, Ueq achieved ▼ Dissipation, Ueq not achieved — Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



Max Depth: 8.025 m / 26.33 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: Every Point

File: 17-05021_SP03.COR

Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010

Coords: UTM 18N N: 5049767m E: 357132m Elev: 149.6m

Sheet No: 1 of 1

- Equilibrium Pore Pressure (U_{eq})

- Assumed Ueq

◀ Dissipation, U_{eq} achieved

◀ Dissipation, U_{eq} not achieved

— Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



Thurber Engineering

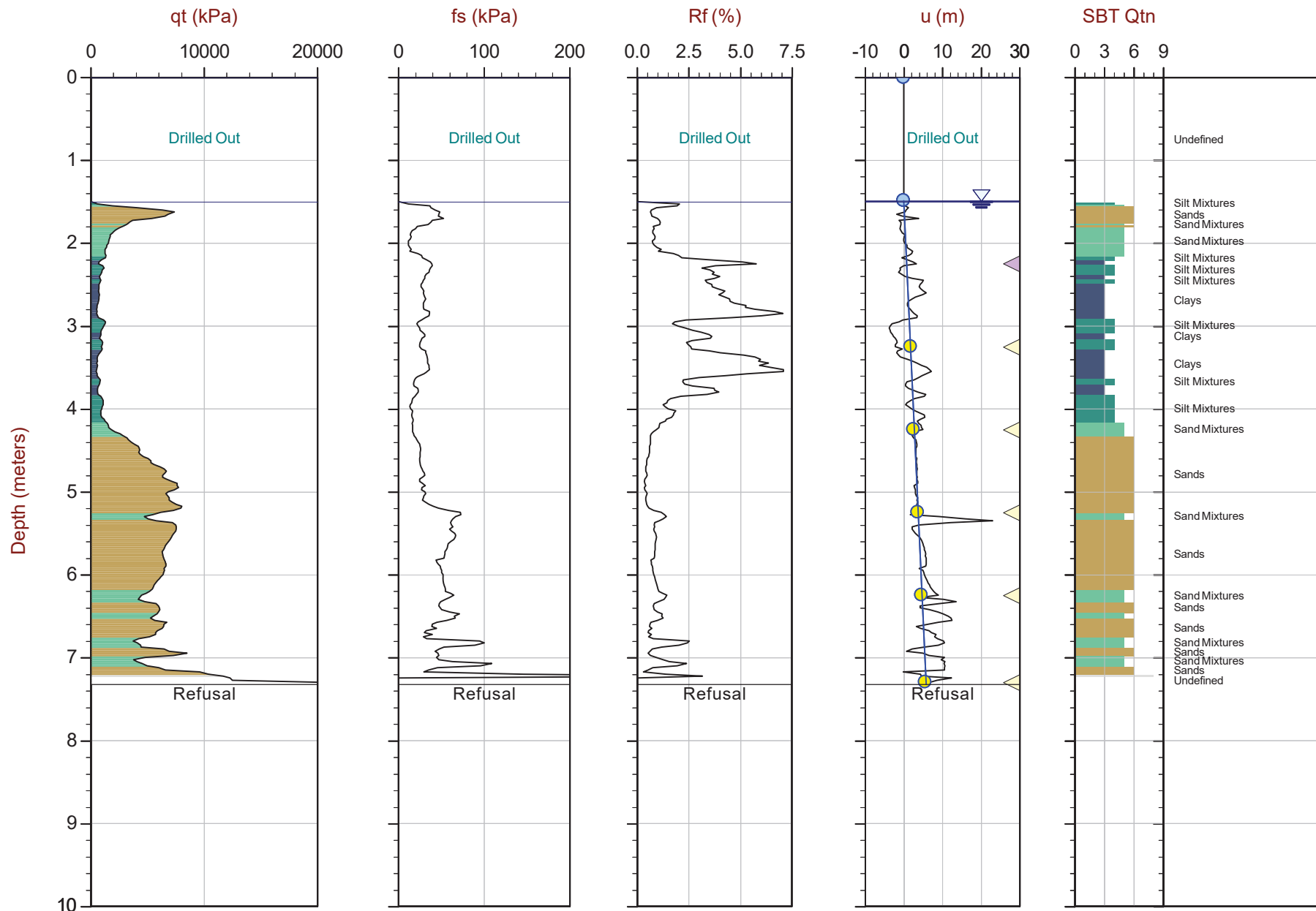
Job No: 17-05021

Date: 2017-05-23 14:17

Site: Hwy 17 - Muskrat Creek Culvert

Sounding: SCPT17-04

Cone: 379:T1500F15U500



Max Depth: 7.325 m / 24.03 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: Every Point

File: 17-05021_SP04.COR

Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010

Coords: UTM 18N N: 5049744m E: 357150m Elev: 149.7m

Sheet No: 1 of 1

● Equilibrium Pore Pressure (Ueq) ● Assumed Ueq ▲ Dissipation, Ueq achieved ▼ Dissipation, Ueq not achieved — Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Advanced Cone Penetration Test Plots with I_c , $S_u(N_{kt})$, and $N1(60)I_c$



Thurber Engineering

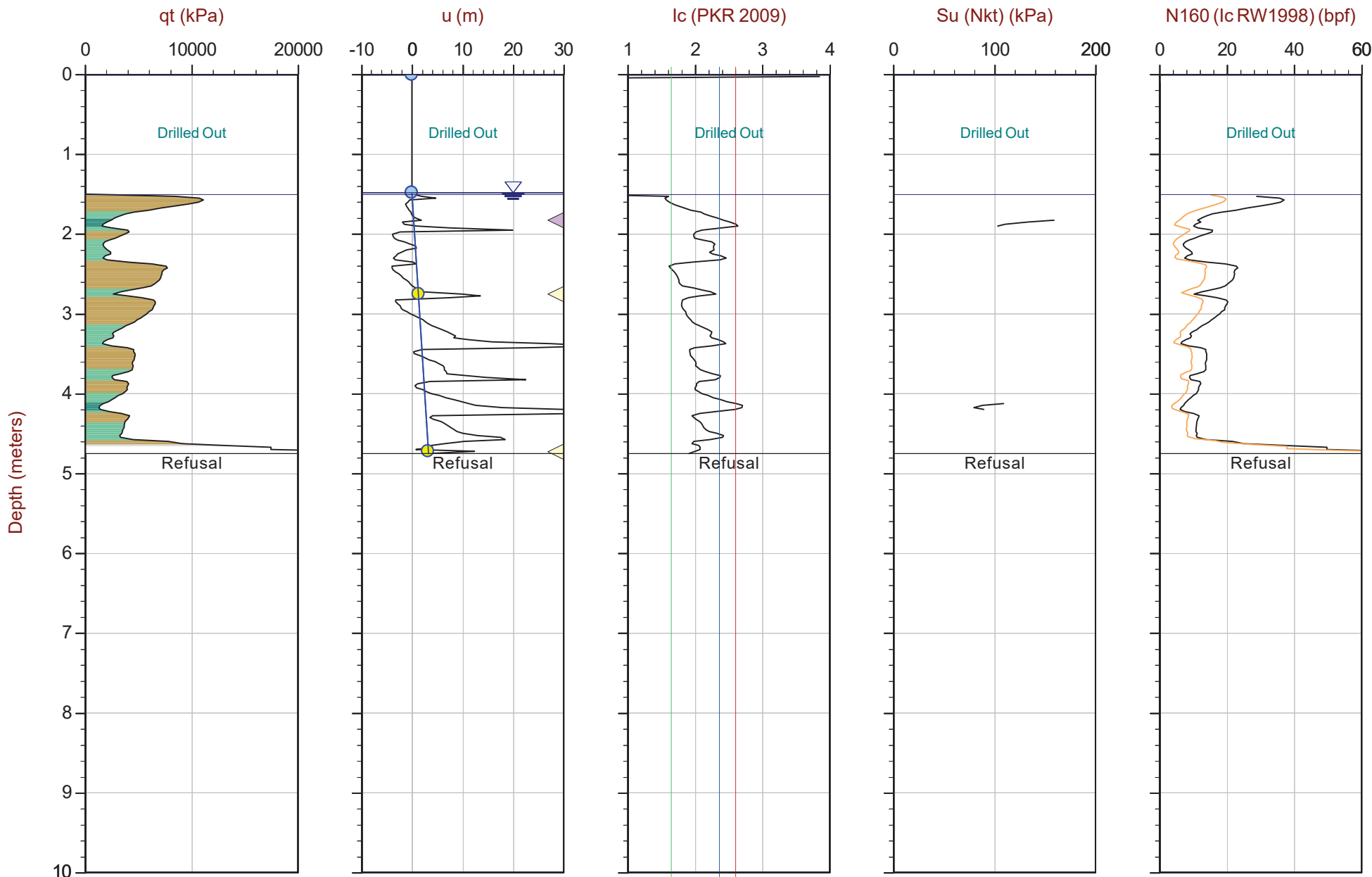
Job No: 17-05021

Date: 2017-05-23 11:34

Site: Hwy 17 - Muskrat Creek Culvert

Sounding: SCPT17-01

Cone: 379:T1500F15U500



Max Depth: 4.750 m / 15.58 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: EveryPoint

File: 17-05021_SP01.COR

Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010

Coords: UTM 18N N: 5049789m E: 357129m Elev: 149.6m

Sheet No: 1 of 1

— N(60) (bpf)

● Equilibrium Pore Pressure (Ueq)

● Assumed Ueq

◀ Dissipation, Ueq achieved

◀ Dissipation, Ueq not achieved

— Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



Thurber Engineering

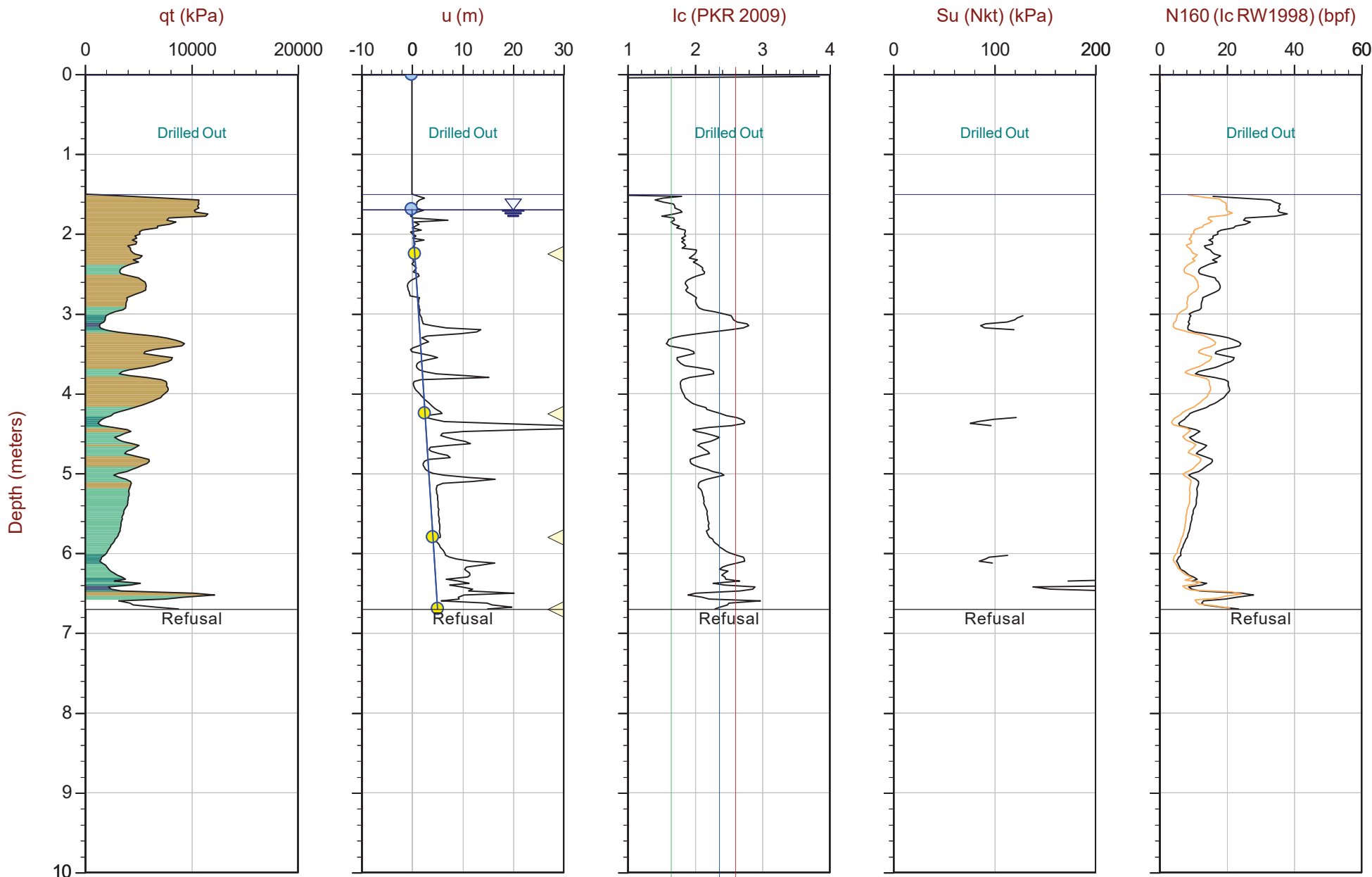
Job No: 17-05021

Date: 2017-05-23 10:19

Site: Hwy 17 - Muskrat Creek Culvert

Sounding: SCPT17-02

Cone: 379:T1500F15U500



Max Depth: 6.700 m / 21.98 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: EveryPoint

File: 17-05021_SP02.COR
Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 18N N: 5049779m E: 357140m Elev: 149.6m
Sheet No: 1 of 1

— N(60) (bpf) ● Equilibrium Pore Pressure (Ueq) ● Assumed Ueq ▲ Dissipation, Ueq achieved ▼ Dissipation, Ueq not achieved — Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



Thurber Engineering

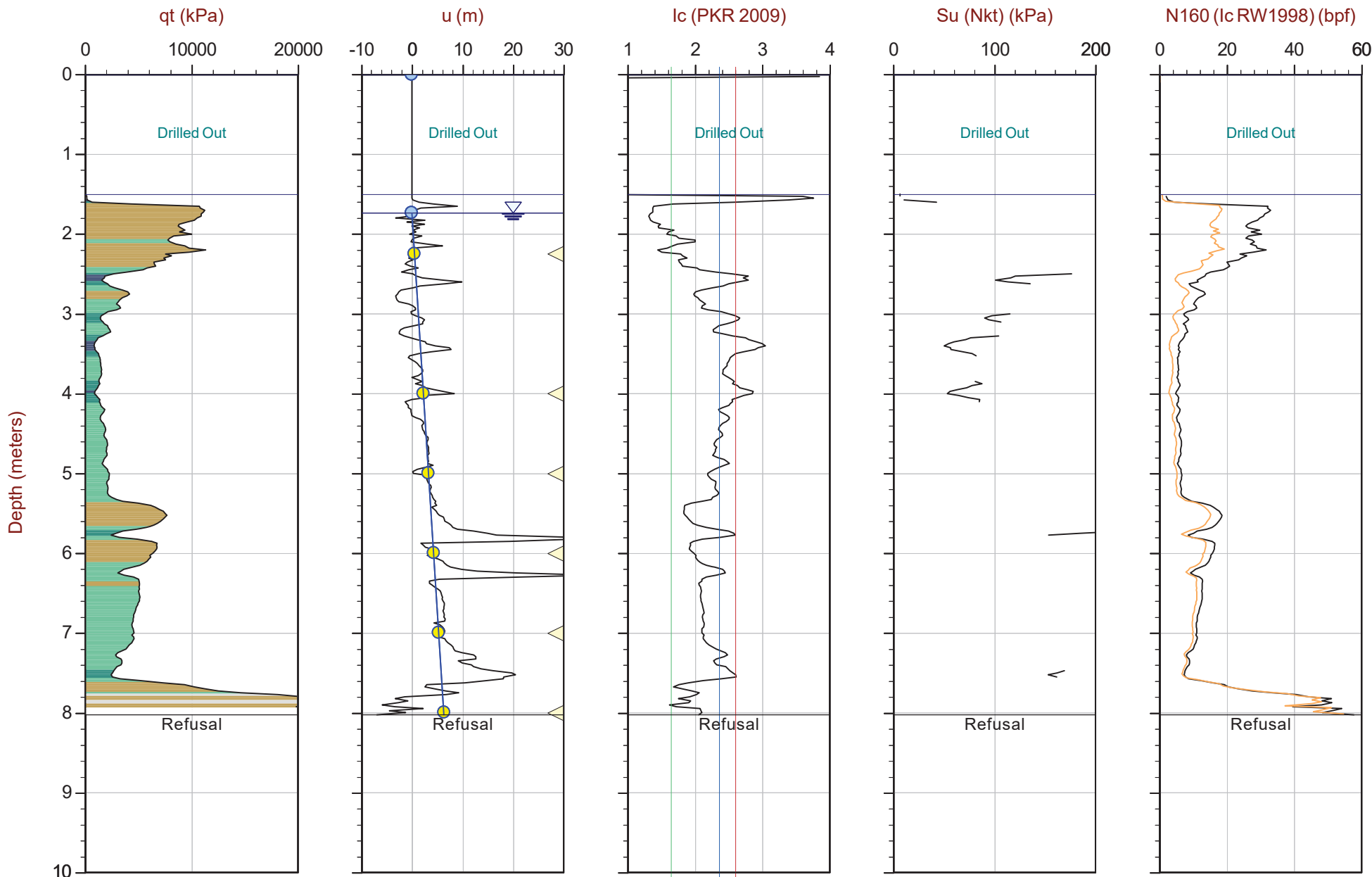
Job No: 17-05021

Date: 2017-05-23 12:56

Site: Hwy 17 - Muskrat Creek Culvert

Sounding: SCPT17-03

Cone: 379:T1500F15U500



Max Depth: 8.025 m / 26.33 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 17-05021_SP03.COR
Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 18N N: 5049767m E: 357132m Elev: 149.6m
Sheet No: 1 of 1

— N(60) (bpf) ● Equilibrium Pore Pressure (Ueq) ● Assumed Ueq ▲ Dissipation, Ueq achieved ▼ Dissipation, Ueq not achieved — Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



Thurber Engineering

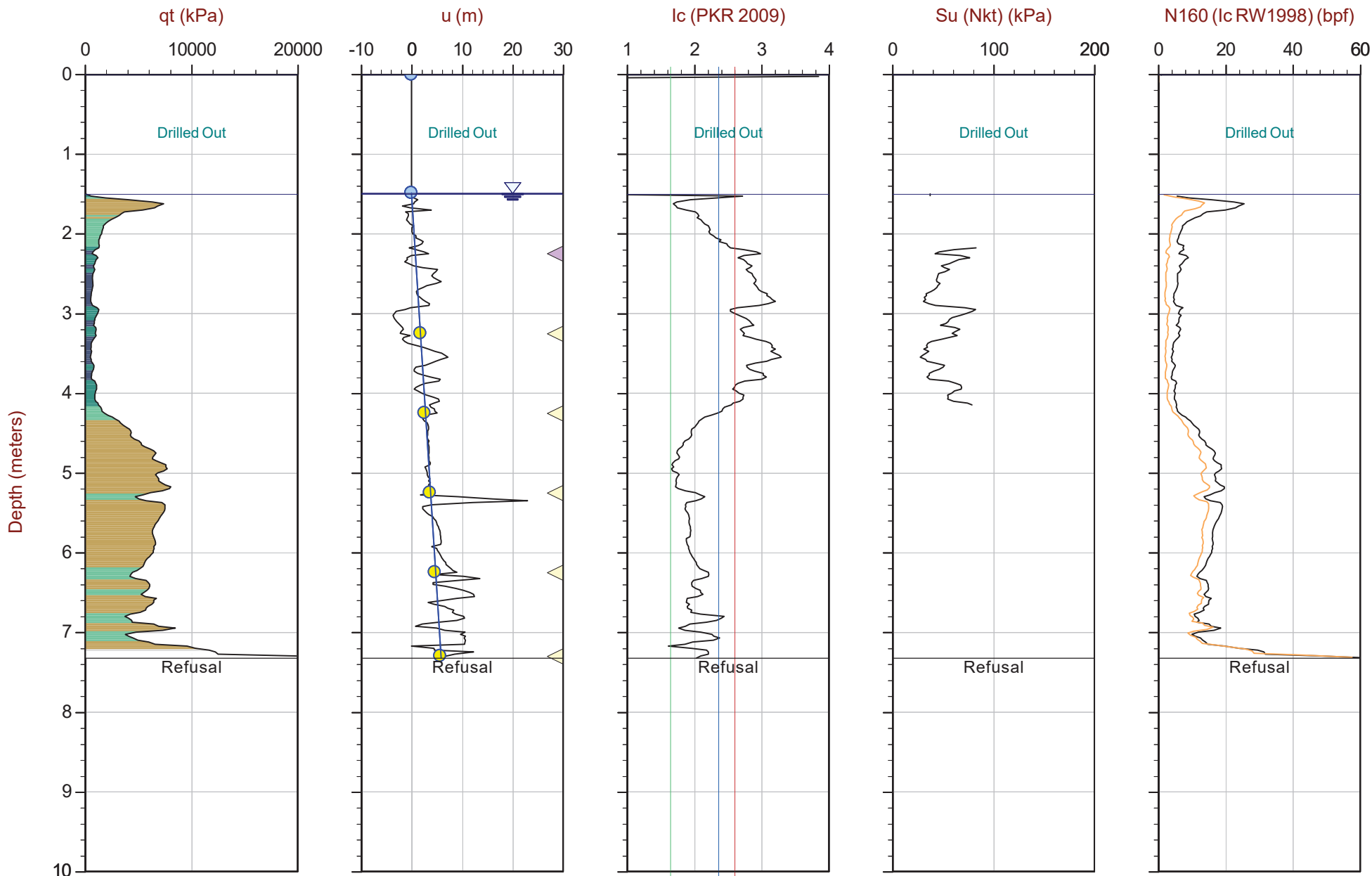
Job No: 17-05021

Date: 2017-05-23 14:17

Site: Hwy 17 - Muskrat Creek Culvert

Sounding: SCPT17-04

Cone: 379:T1500F15U500



Max Depth: 7.325 m / 24.03 ft

Depth Inc: 0.025 m / 0.082 ft

Avg Int: EveryPoint

File: 17-05021_SP04.COR

Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010

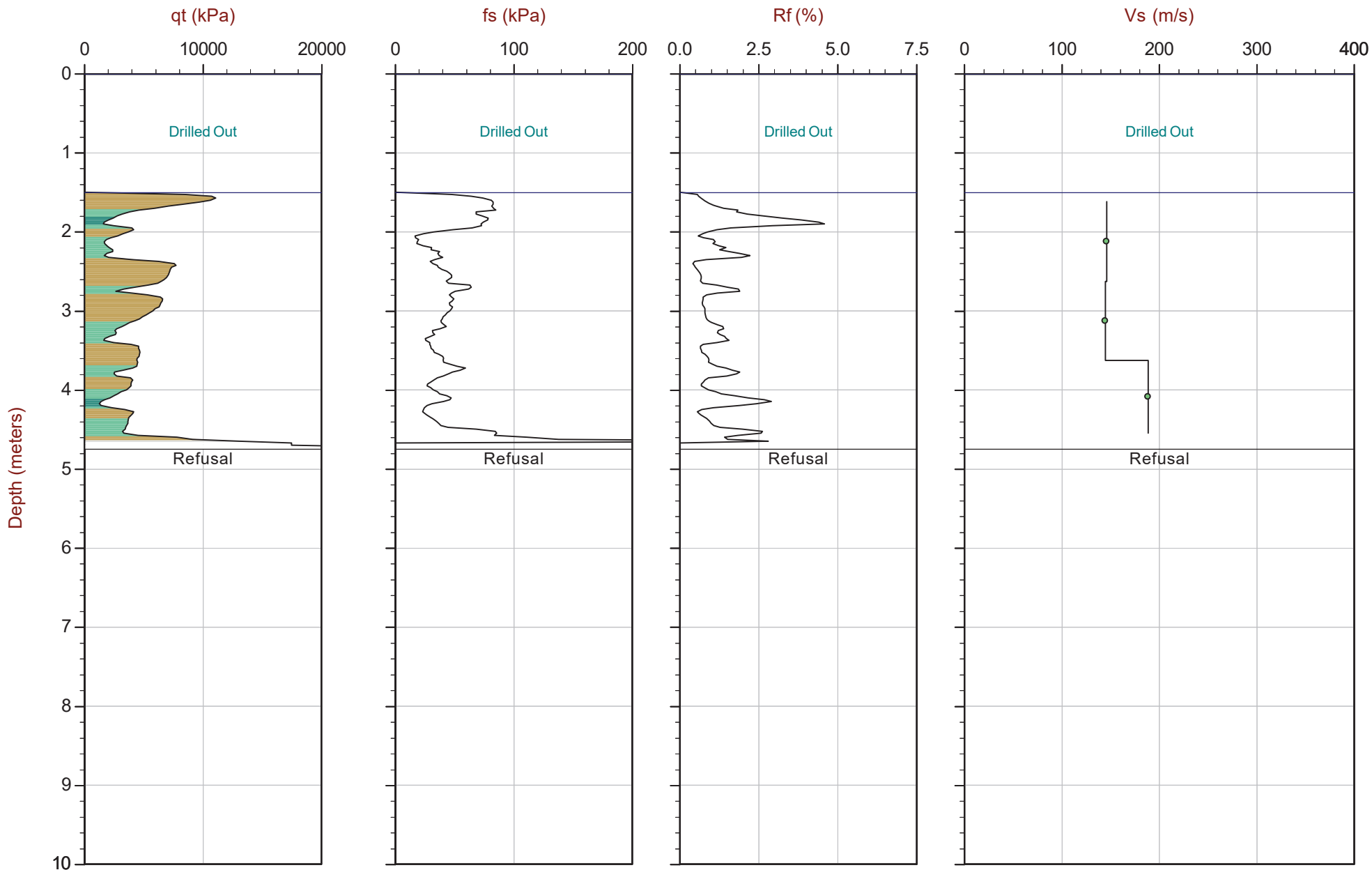
Coords: UTM 18N N: 5049744m E: 357150m Elev: 149.7m

Sheet No: 1 of 1

— N(60) (bpf) ● Equilibrium Pore Pressure (Ueq) ● Assumed Ueq ▲ Dissipation, Ueq achieved ▼ Dissipation, Ueq not achieved — Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Seismic Cone Penetration Test Plots



Max Depth: 4.750 m / 15.58 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: EveryPoint

File: 17-05021_SP01.COR
Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 18N N: 5049789m E: 357129m Elev: 149.6m
Sheet No: 1 of 1



Thurber Engineering

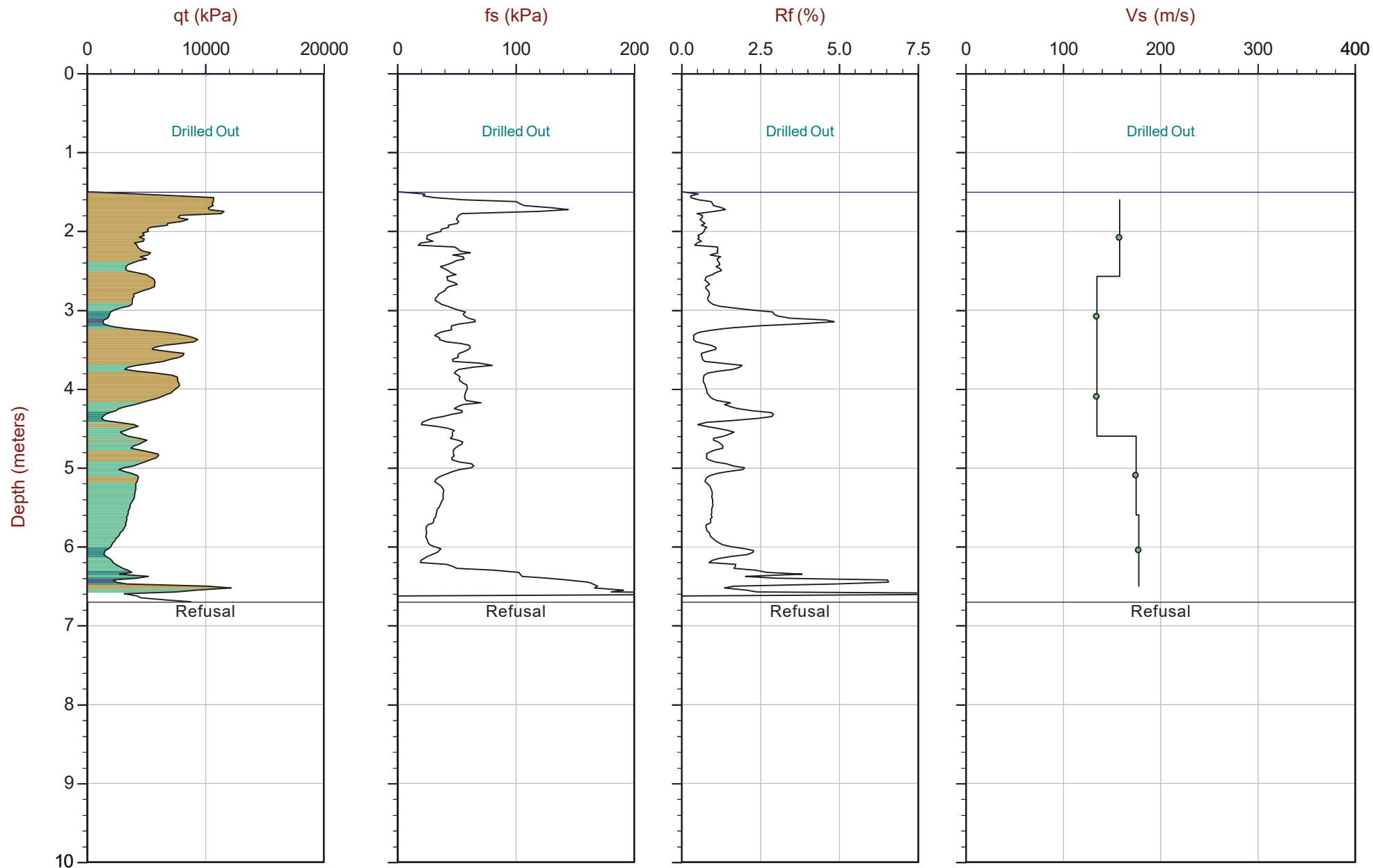
Job No: 17-05021

Date: 2017-05-23 10:19

Site: Hwy 17 - Muskrat Creek Culvert

Sounding: SCPT17-02

Cone: 379:T1500F15U500



Max Depth: 6.700 m / 21.98 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 17-05021_SP02.COR
Unit Wt: SBTQtn(PKR2009)

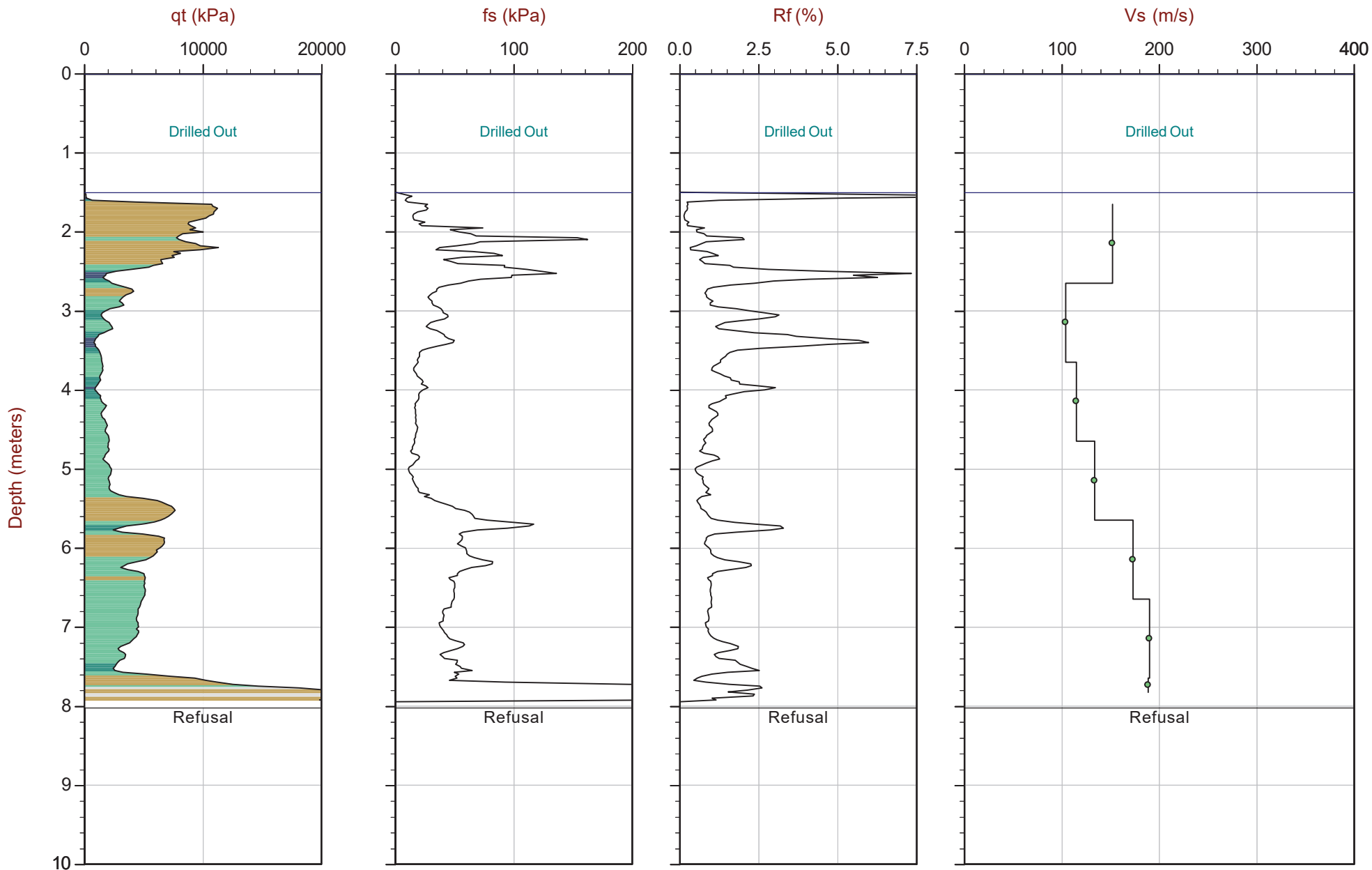
SBT: Robertson, 2009 and 2010
Coords: UTM 18N N: 5049779m E: 357140m Elev: 149.6m
Sheet No: 1 of 1



Thurber Engineering

Job No: 17-05021
Date: 2017-05-23 12:56
Site: Hwy 17 - Muskrat Creek Culvert

Sounding: SCPT17-03
Cone: 379:T1500F15U500



Max Depth: 8.025 m / 26.33 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: EveryPoint

File: 17-05021_SP03.COR
Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 18N N: 5049767m E: 357132m Elev: 149.6m
Sheet No: 1 of 1



Thurber Engineering

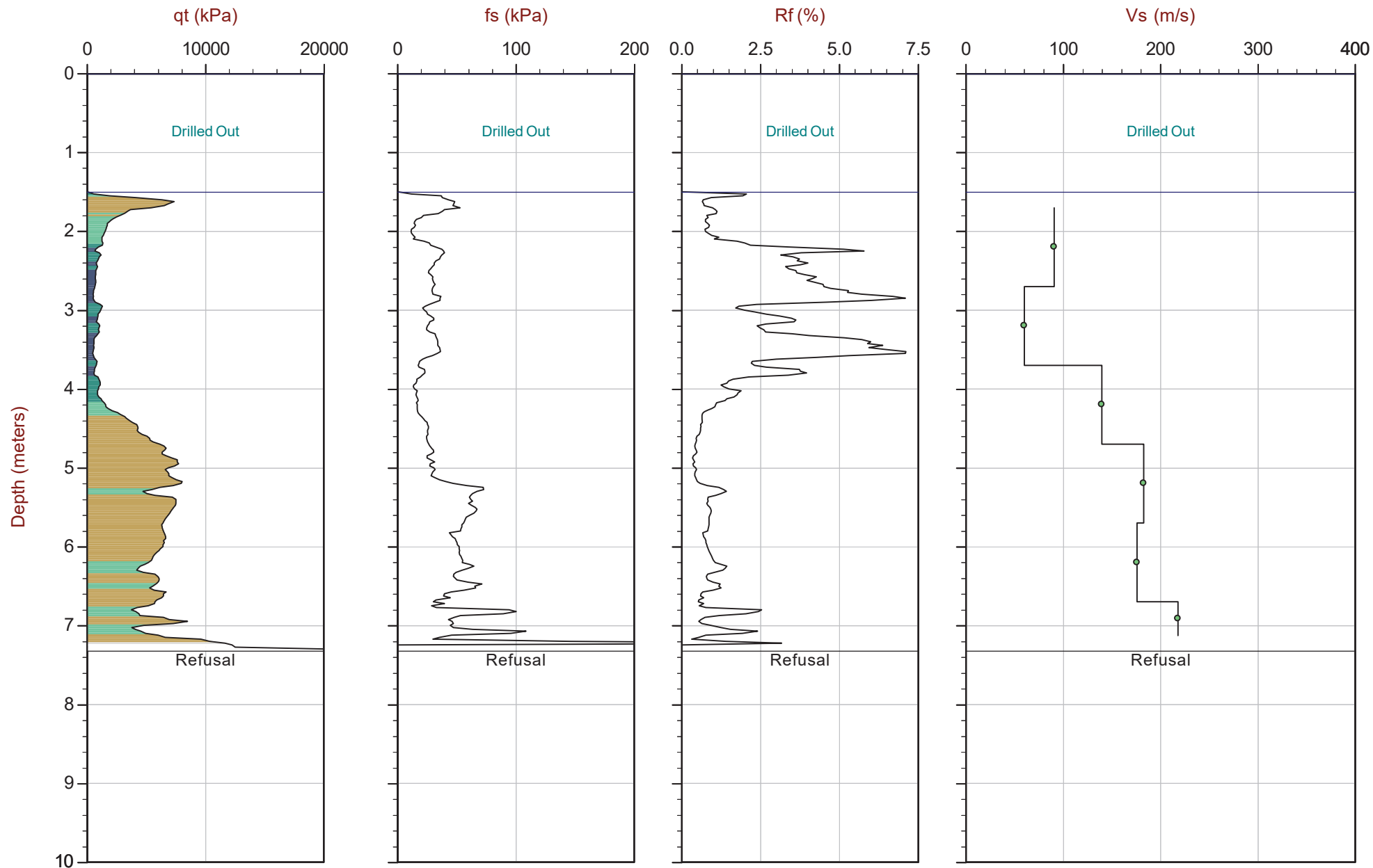
Job No: 17-05021

Date: 2017-05-23 14:17

Site: Hwy 17 - Muskrat Creek Culvert

Sounding: SCPT17-04

Cone: 379:T1500F15U500



Max Depth: 7.325 m / 24.03 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 17-05021_SP04.COR
Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 18N N: 5049744m E: 357150m Elev: 149.7m
Sheet No: 1 of 1

Seismic Cone Penetration Test Tabular Results



Job No: 17-05021
Client: Thurber Engineering
Project: Hwy 17 - Muskrat Creek Culvert
Sounding ID: SCPT17-01
Date: 23-May-2017

Seismic Source: Beam
Source Offset (m): 0.55
Source Depth (m): 0.00
Geophone Offset (m): 0.20

SCPT_u SHEAR WAVE VELOCITY TEST RESULTS - V_s

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
1.82	1.62	1.71			
2.83	2.63	2.69	0.98	6.69	146
3.83	3.63	3.67	0.98	6.80	145
4.75	4.55	4.58	0.91	4.82	189



Job No: 17-05021
Client: Thurber Engineering
Project: Hwy 17 - Muskrat Creek Culvert
Sounding ID: SCPT17-02
Date: 23-May-2017

Seismic Source: Beam
Source Offset (m): 0.55
Source Depth (m): 0.00
Geophone Offset (m): 0.20

SCPT_u SHEAR WAVE VELOCITY TEST RESULTS - V_s

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
1.80	1.60	1.69			
2.77	2.57	2.63	0.94	5.92	158
3.80	3.60	3.64	1.01	7.51	135
4.80	4.60	4.63	0.99	7.33	135
5.80	5.60	5.63	0.99	5.69	175
6.70	6.50	6.52	0.90	5.03	178



Job No: 17-05021
Client: Thurber Engineering
Project: Hwy 17 - Muskrat Creek Culvert
Sounding ID: SCPT17-03
Date: 23-May-2017

Seismic Source: Beam
Source Offset (m): 0.55
Source Depth (m): 0.00
Geophone Offset (m): 0.20

SCPT_u SHEAR WAVE VELOCITY TEST RESULTS - V_s

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
1.85	1.65	1.74			
2.85	2.65	2.71	0.97	6.35	152
3.85	3.65	3.69	0.98	9.45	104
4.85	4.65	4.68	0.99	8.60	115
5.85	5.65	5.68	0.99	7.43	134
6.85	6.65	6.67	1.00	5.76	173
7.85	7.65	7.67	1.00	5.25	190
8.03	7.83	7.85	0.18	0.95	189



Job No: 17-05021
Client: Thurber Engineering
Project: Hwy 17 - Muskrat Creek Culvert
Sounding ID: SCPT17-04
Date: 23-May-2017

Seismic Source: Beam
Source Offset (m): 0.55
Source Depth (m): 0.00
Geophone Offset (m): 0.20

SCPT_u SHEAR WAVE VELOCITY TEST RESULTS - V_s

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
1.90	1.70	1.79			
2.90	2.70	2.76	0.97	10.69	91
3.90	3.70	3.74	0.99	16.37	60
4.90	4.70	4.73	0.99	7.11	140
5.90	5.70	5.73	0.99	5.43	183
6.90	6.70	6.72	1.00	5.65	176
7.33	7.13	7.15	0.43	1.97	218

Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots



Job No: 17-05021
Client: Thurber Engineering
Project: Hwy 17 - Muskrat Creek Culvert
Start Date: 23-May-2017
End Date: 23-May-2017

CPTu PORE PRESSURE DISSIPATION SUMMARY

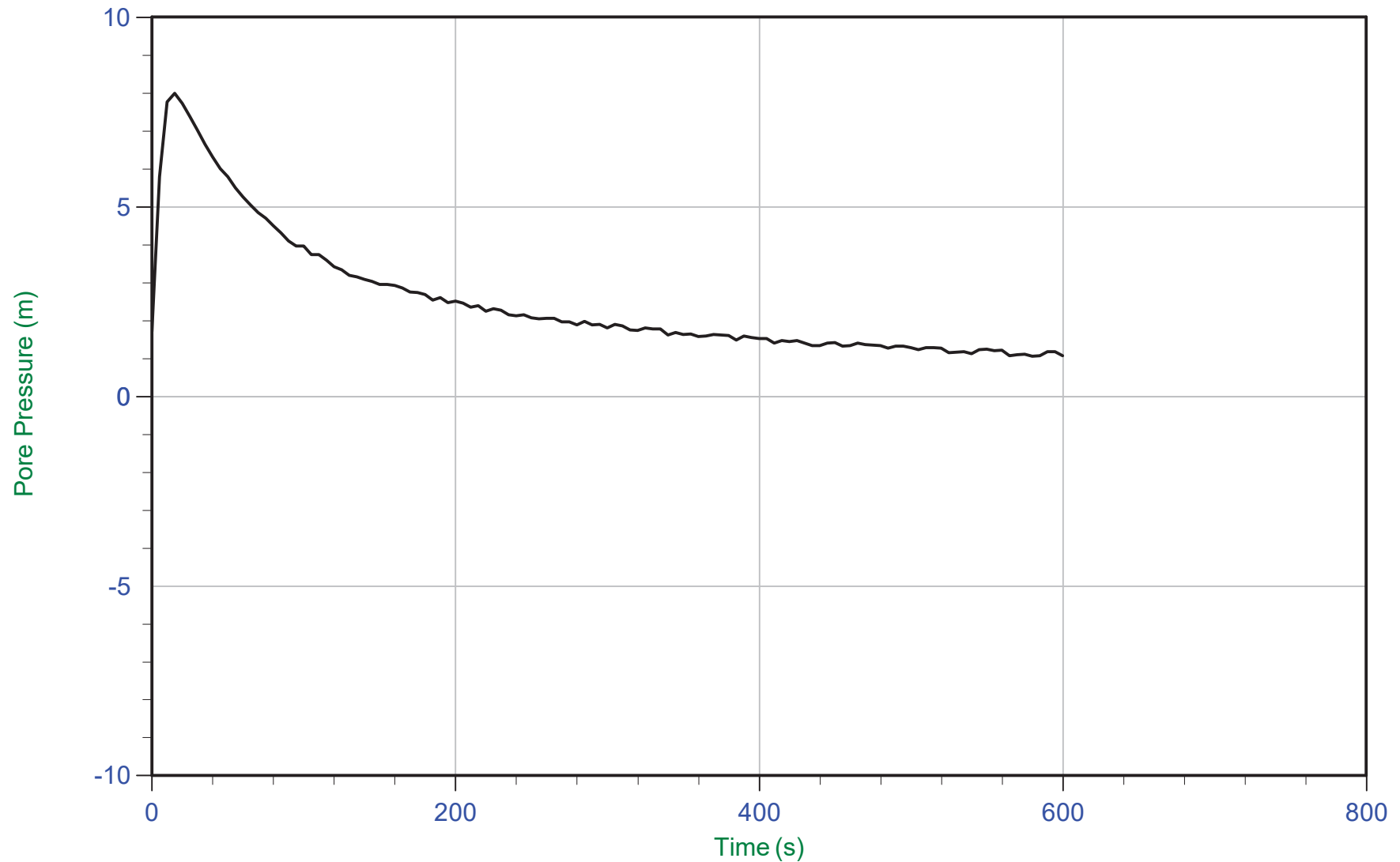
Sounding ID	File Name	Cone Area (cm ²)	Duration (s)	Test Depth (m)	Estimated Equilibrium Pore Pressure U _{eq} (m)	Calculated Phreatic Surface (m)
SCPT17-01	17-05021_SP01	15	600	1.825	Not Achieved	
SCPT17-01	17-05021_SP01	15	500	2.750	1.3	1.5
SCPT17-01	17-05021_SP01	15	350	4.725	3.1	1.6
SCPT17-02	17-05021_SP02	15	200	2.250	0.6	1.7
SCPT17-02	17-05021_SP02	15	250	4.250	2.6	1.7
SCPT17-02	17-05021_SP02	15	350	5.800	4.2	1.7
SCPT17-02	17-05021_SP02	15	300	6.700	5.1	1.6
SCPT17-03	17-05021_SP03	15	300	2.250	0.5	1.7
SCPT17-03	17-05021_SP03	15	350	4.000	2.3	1.7
SCPT17-03	17-05021_SP03	15	300	5.000	3.3	1.7
SCPT17-03	17-05021_SP03	15	250	6.000	4.3	1.7
SCPT17-03	17-05021_SP03	15	250	7.000	5.3	1.7
SCPT17-03	17-05021_SP03	15	250	8.000	6.4	1.6
SCPT17-04	17-05021_SP04	15	310	2.250	Not Achieved	
SCPT17-04	17-05021_SP04	15	450	3.250	1.8	1.5
SCPT17-04	17-05021_SP04	15	250	4.250	2.6	1.7
SCPT17-04	17-05021_SP04	15	300	5.250	3.6	1.6
SCPT17-04	17-05021_SP04	15	200	6.250	4.6	1.7
SCPT17-04	17-05021_SP04	15	300	7.300	5.6	1.7



Thurber Engineering

Job No: 17-05021
Date: 05/23/2017 11:34
Site: Hwy 17 - Muskrat Creek Culvert

Sounding: SCPT17-01
Cone: 379:T1500F15U500 Area=15 cm²



Trace Summary: Filename: 17-05021_SP01.PPF U Min: 1.1 m
 Depth: 1.825 m / 5.987 ft U Max: 8.0 m
 Duration: 600.0 s



Thurber Engineering

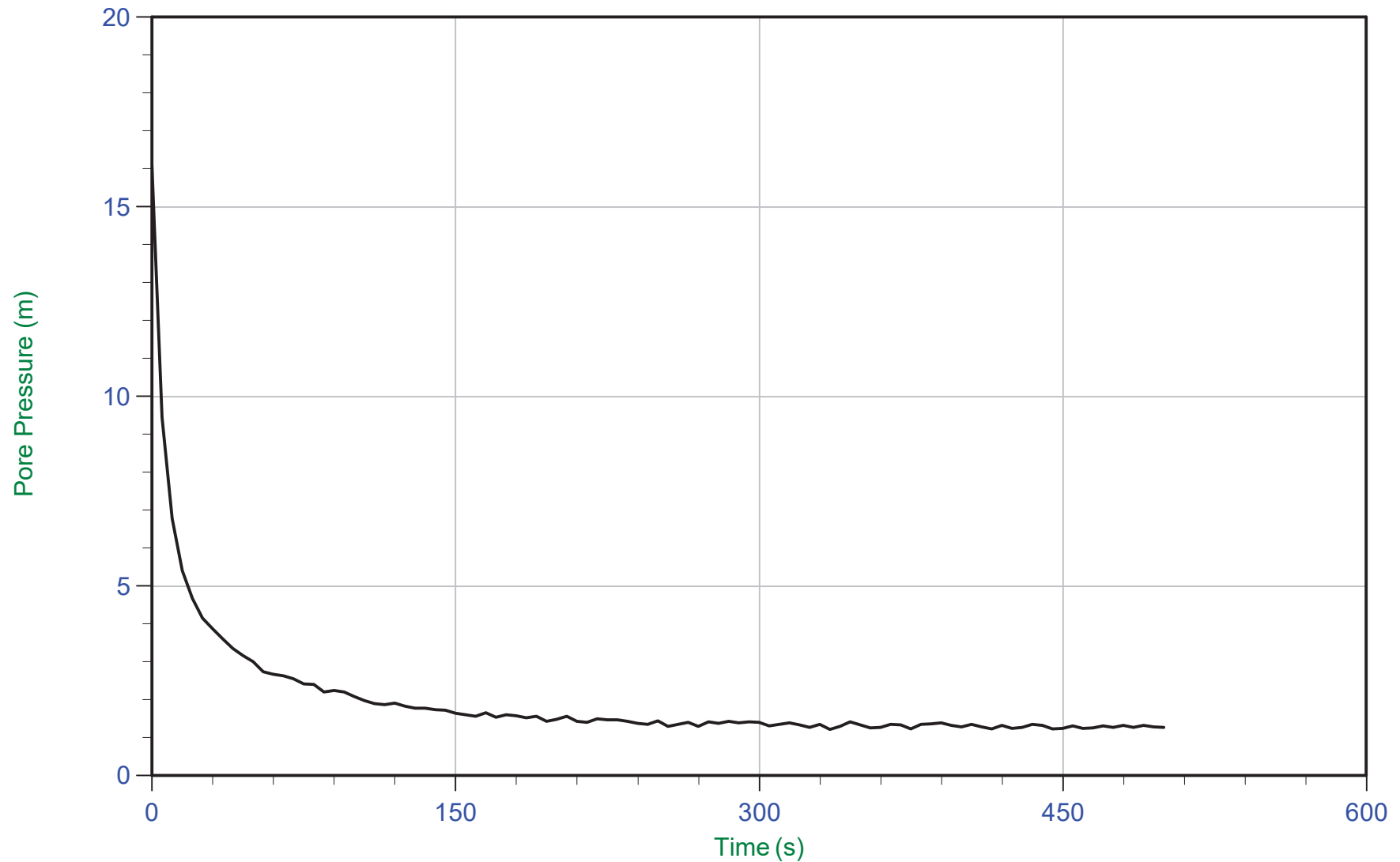
Job No: 17-05021

Date: 05/23/2017 11:34

Site: Hwy 17 - Muskrat Creek Culvert

Sounding: SCPT17-01

Cone: 379:T1500F15U500 Area=15 cm²



Trace Summary: Filename: 17-05021_SP01.PPF
Depth: 2.750 m / 9.022 ft
Duration: 500.0 s

U Min: 1.2 m
U Max: 16.1 m

WT: 1.481 m / 4.859 ft
Ueq: 1.3 m



Thurber Engineering

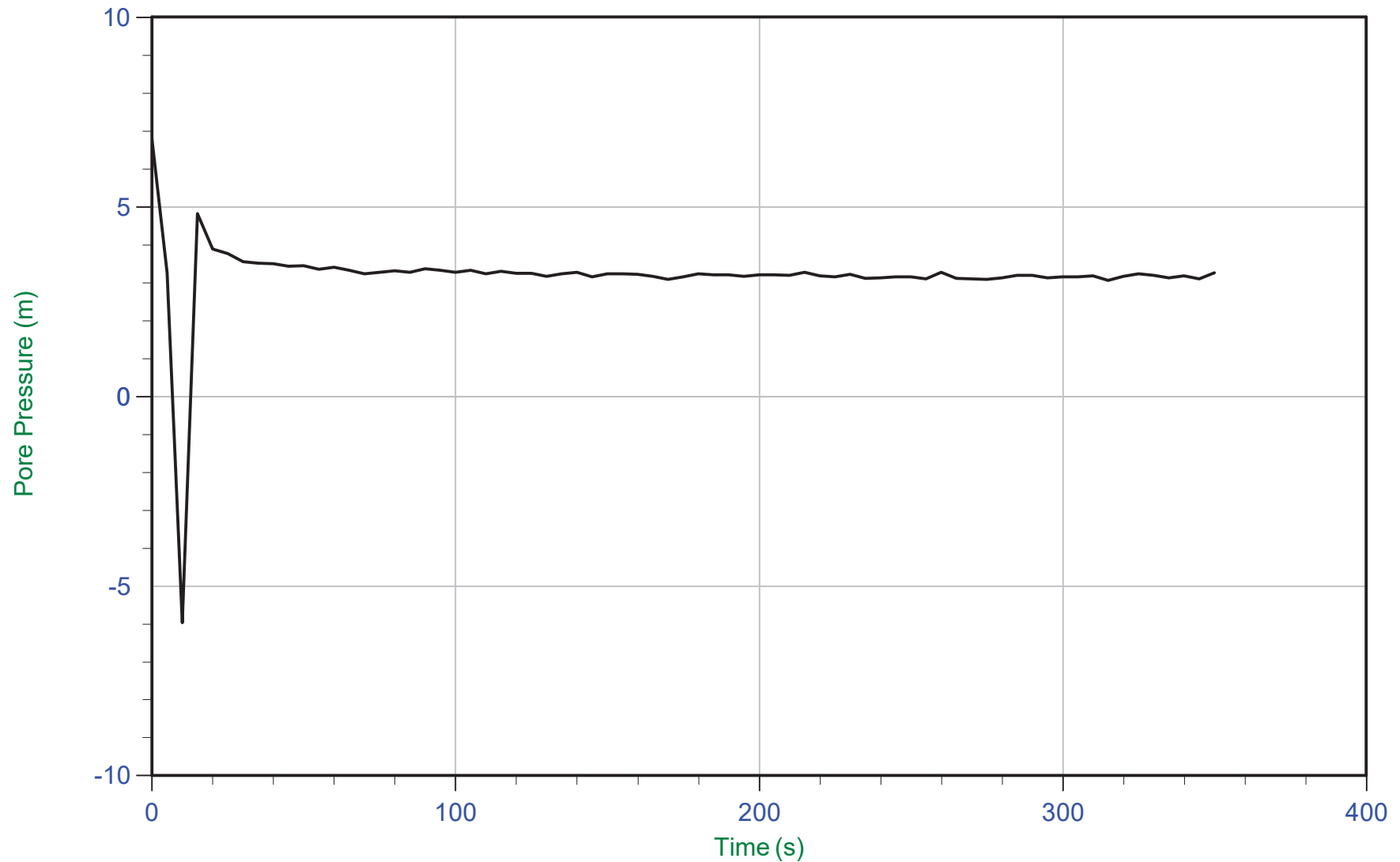
Job No: 17-05021

Date: 05/23/2017 11:34

Site: Hwy 17 - Muskrat Creek Culvert

Sounding: SCPT17-01

Cone: 379:T1500F15U500 Area=15 cm²



Trace Summary: Filename: 17-05021_SP01.PPF
Depth: 4.725 m / 15.502 ft
Duration: 350.0 s

U Min: -6.0 m
U Max: 6.8 m

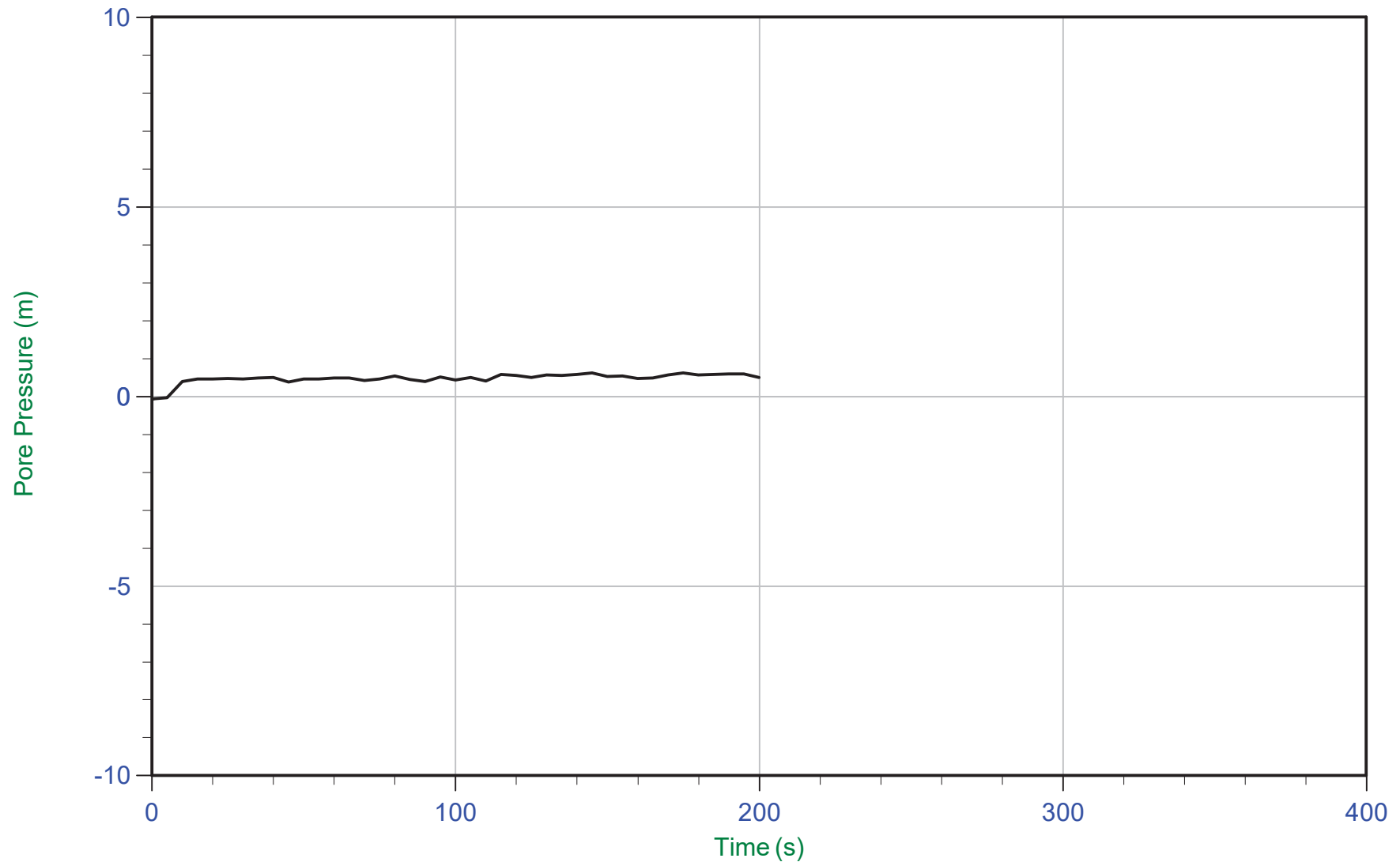
WT: 1.585 m / 5.200 ft
Ueq: 3.1 m



Thurber Engineering

Job No: 17-05021
Date: 05/23/2017 10:19
Site: Hwy 17 - Muskrat Creek Culvert

Sounding: SCPT17-02
Cone: 379:T1500F15U500 Area=15 cm²



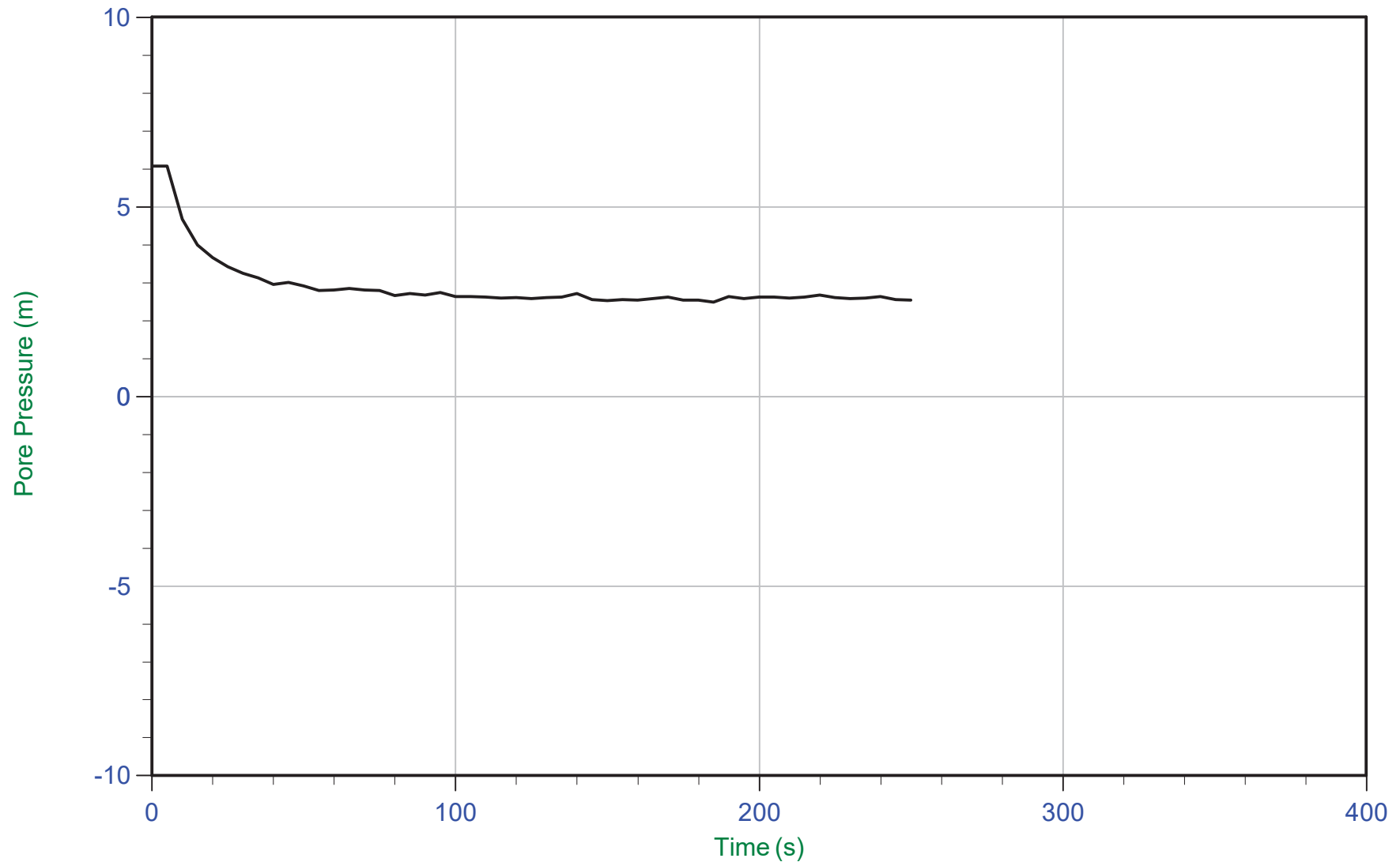
Trace Summary:	Filename: 17-05021_SP02.PPF	U Min: -0.1 m	WT: 1.693 m / 5.554 ft
	Depth: 2.250 m / 7.382 ft	U Max: 0.6 m	Ueq: 0.6 m
	Duration: 200.0 s		



Thurber Engineering

Job No: 17-05021
Date: 05/23/2017 10:19
Site: Hwy 17 - Muskrat Creek Culvert

Sounding: SCPT17-02
Cone: 379:T1500F15U500 Area=15 cm²



Trace Summary:	Filename: 17-05021_SP02.PPF	U Min: 2.5 m	WT: 1.675 m / 5.495 ft
	Depth: 4.250 m / 13.943 ft	U Max: 6.1 m	Ueq: 2.6 m
	Duration: 250.0 s		



Thurber Engineering

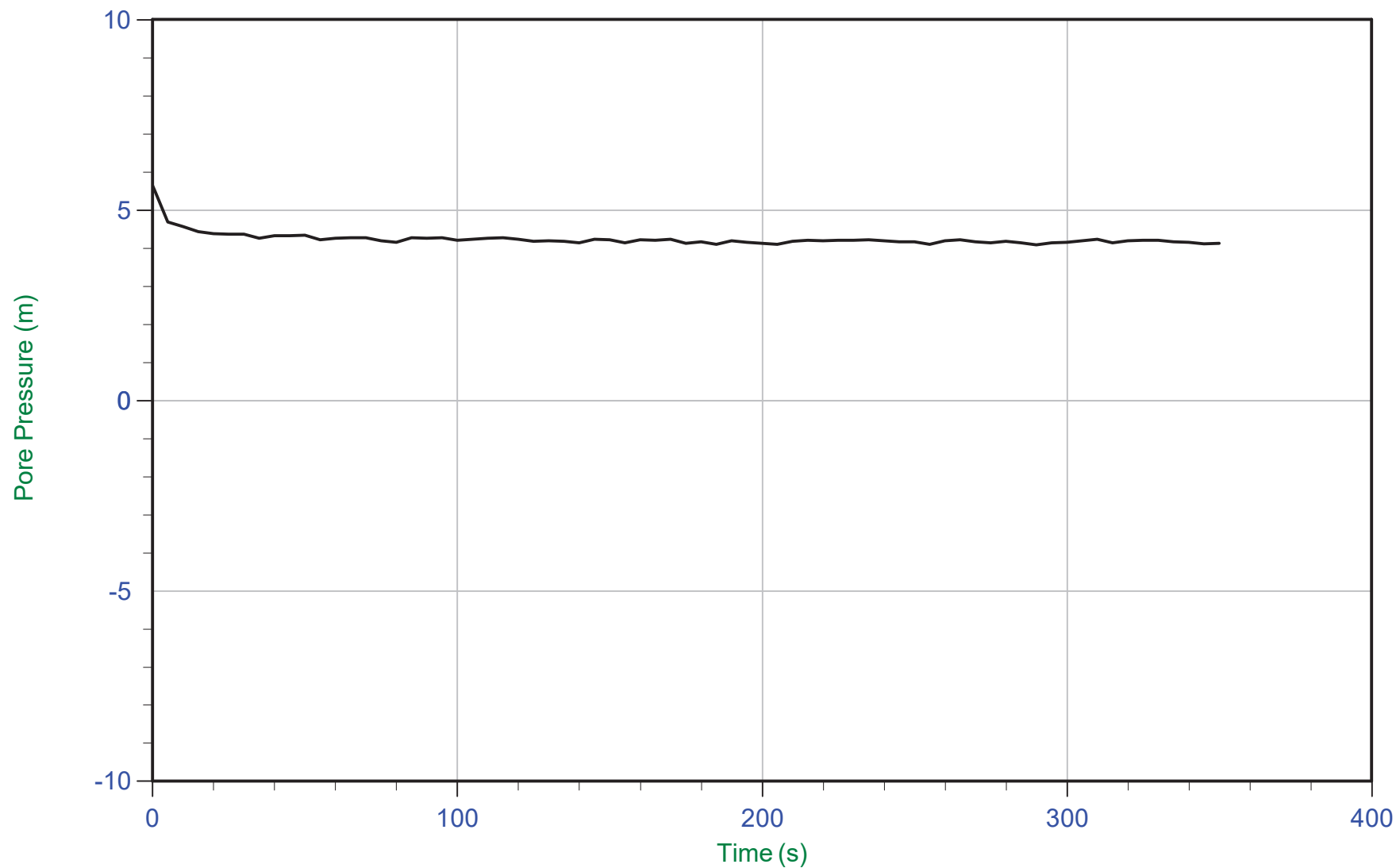
Job No: 17-05021

Date: 05/23/2017 10:19

Site: Hwy 17 - Muskrat Creek Culvert

Sounding: SCPT17-02

Cone: 379:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 17-05021_SP02.PPF

Depth: 5.800 m / 19.029 ft

Duration: 350.0 s

U Min: 4.1 m

U Max: 5.6 m

WT: 1.648 m / 5.407 ft

Ueq: 4.2 m



Thurber Engineering

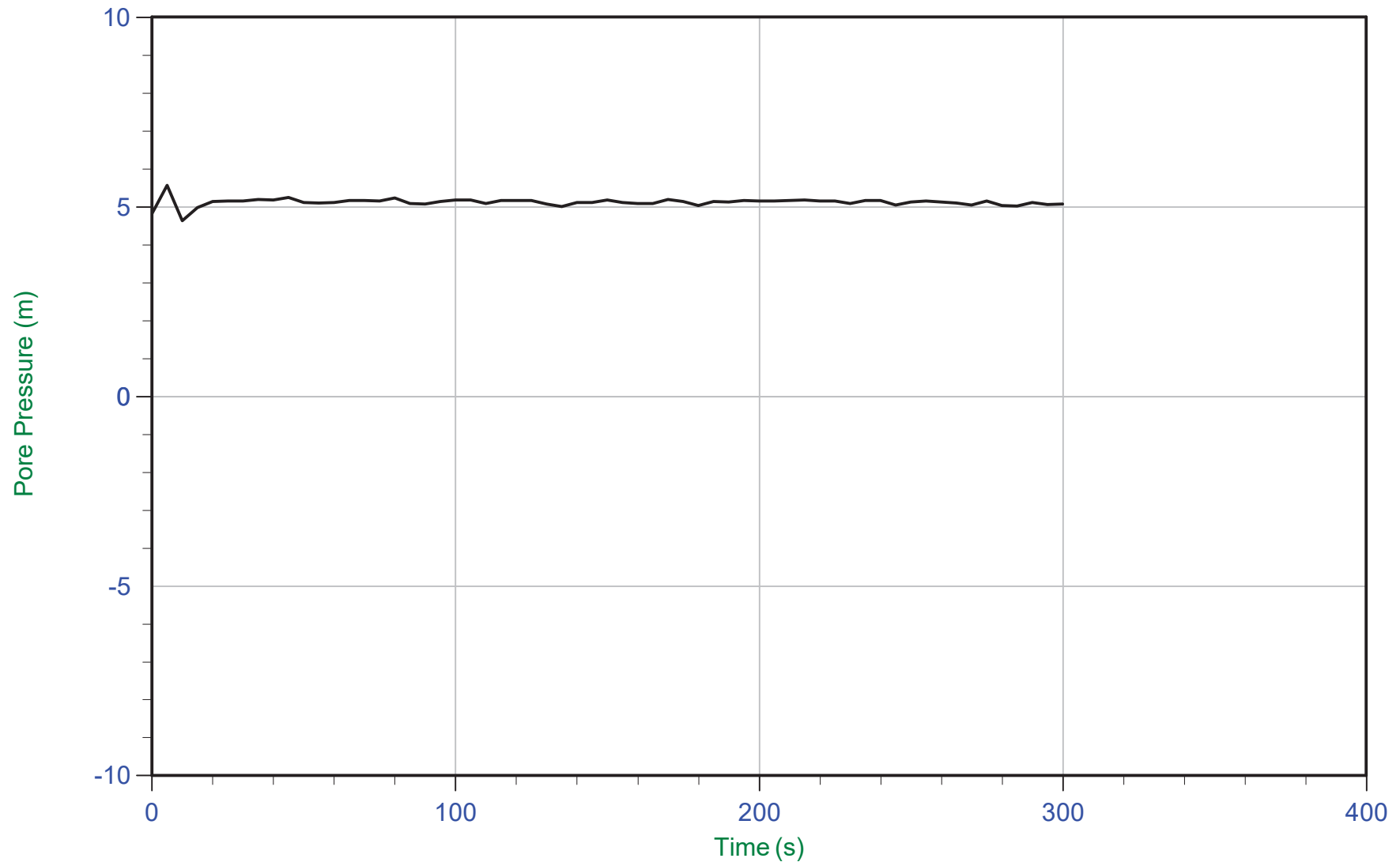
Job No: 17-05021

Date: 05/23/2017 10:19

Site: Hwy 17 - Muskrat Creek Culvert

Sounding: SCPT17-02

Cone: 379:T1500F15U500 Area=15 cm²



Trace Summary: Filename: 17-05021_SP02.PPF
Depth: 6.700 m / 21.981 ft
Duration: 300.0 s

U Min: 4.6 m
U Max: 5.6 m

WT: 1.603 m / 5.259 ft
Ueq: 5.1 m



Thurber Engineering

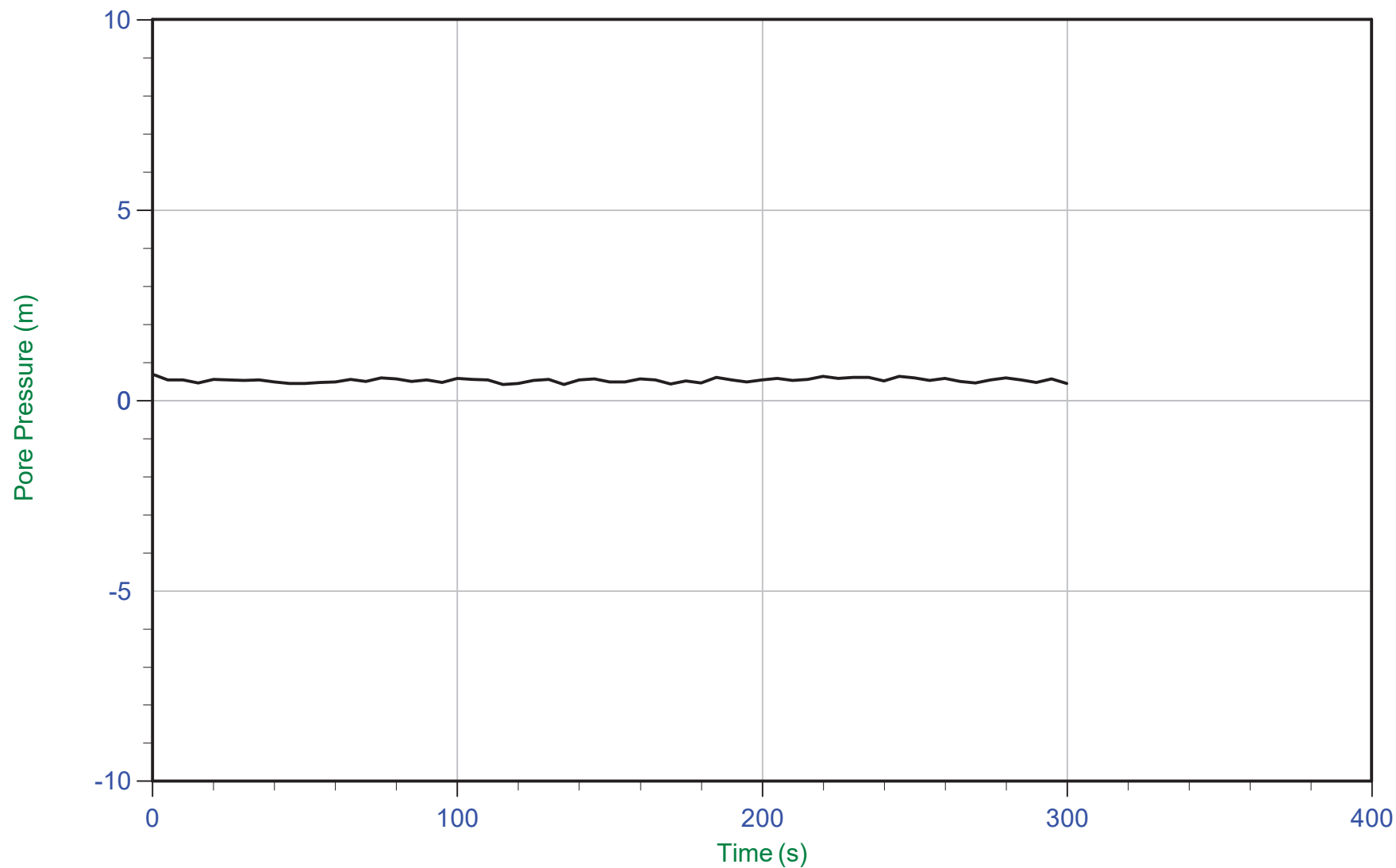
Job No: 17-05021

Date: 05/23/2017 12:56

Site: Hwy 17 - Muskrat Creek Culvert

Sounding: SCPT17-03

Cone: 379:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 17-05021_SP03.PPF

Depth: 2.250 m / 7.382 ft

Duration: 300.0 s

U Min: 0.4 m

U Max: 0.7 m

WT: 1.734 m / 5.689 ft

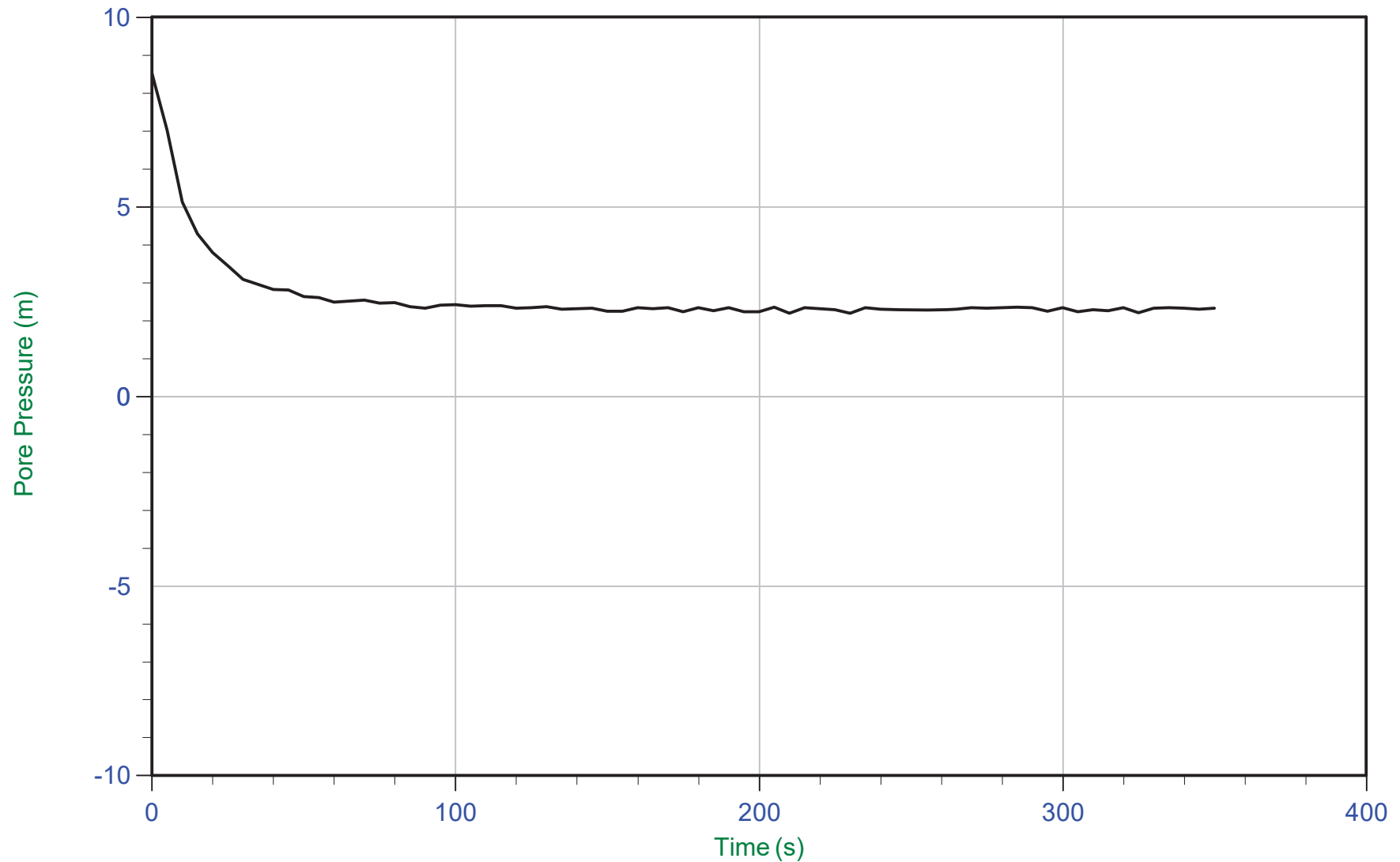
Ueq: 0.5 m



Thurber Engineering

Job No: 17-05021
Date: 05/23/2017 12:56
Site: Hwy 17 - Muskrat Creek Culvert

Sounding: SCPT17-03
Cone: 379:T1500F15U500 Area=15 cm²



Trace Summary:	Filename: 17-05021_SP03.PPF	U Min: 2.2 m	WT: 1.731 m / 5.679 ft
	Depth: 4.000 m / 13.123 ft	U Max: 8.5 m	Ueq: 2.3 m
	Duration: 350.0 s		



Thurber Engineering

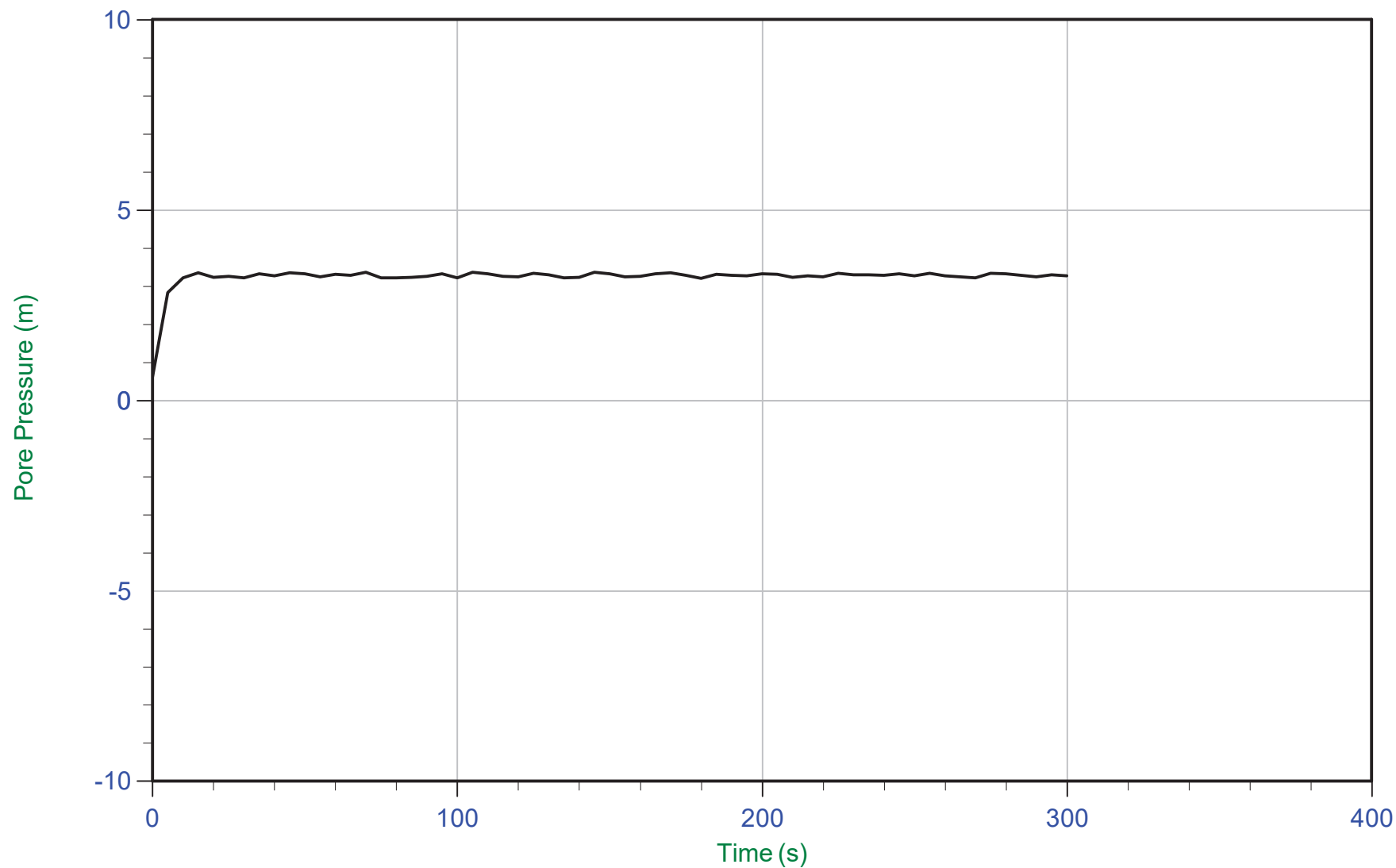
Job No: 17-05021

Date: 05/23/2017 12:56

Site: Hwy 17 - Muskrat Creek Culvert

Sounding: SCPT17-03

Cone: 379:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 17-05021_SP03.PPF

Depth: 5.000 m / 16.404 ft

Duration: 300.0 s

U Min: 0.6 m

U Max: 3.4 m

WT: 1.713 m / 5.620 ft

Ueq: 3.3 m



Thurber Engineering

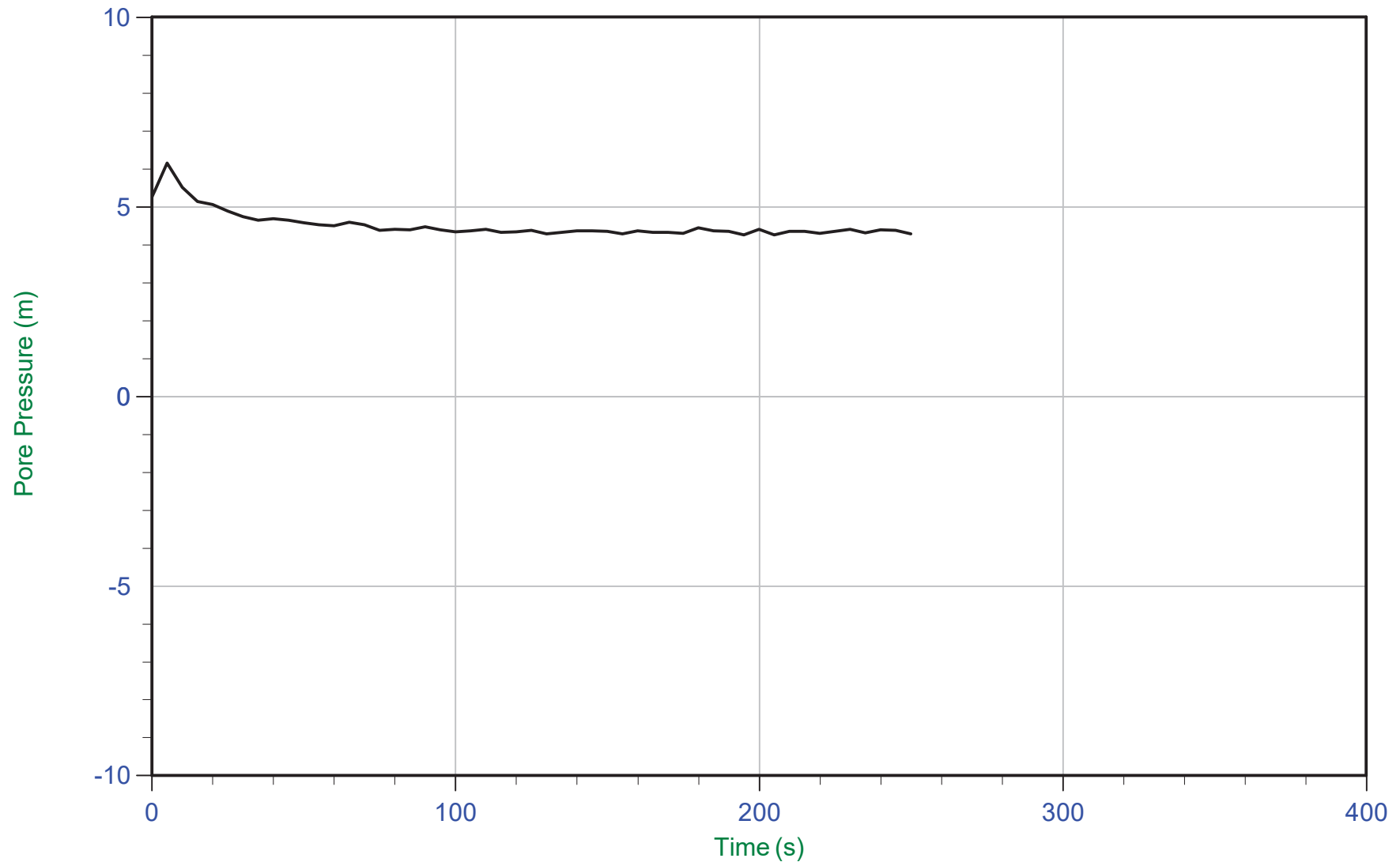
Job No: 17-05021

Date: 05/23/2017 12:56

Site: Hwy 17 - Muskrat Creek Culvert

Sounding: SCPT17-03

Cone: 379:T1500F15U500 Area=15 cm²



Trace Summary:	Filename: 17-05021_SP03.PPF	U Min: 4.3 m	WT: 1.665 m / 5.463 ft
	Depth: 6.000 m / 19.685 ft	U Max: 6.2 m	Ueq: 4.3 m
	Duration: 250.0 s		



Thurber Engineering

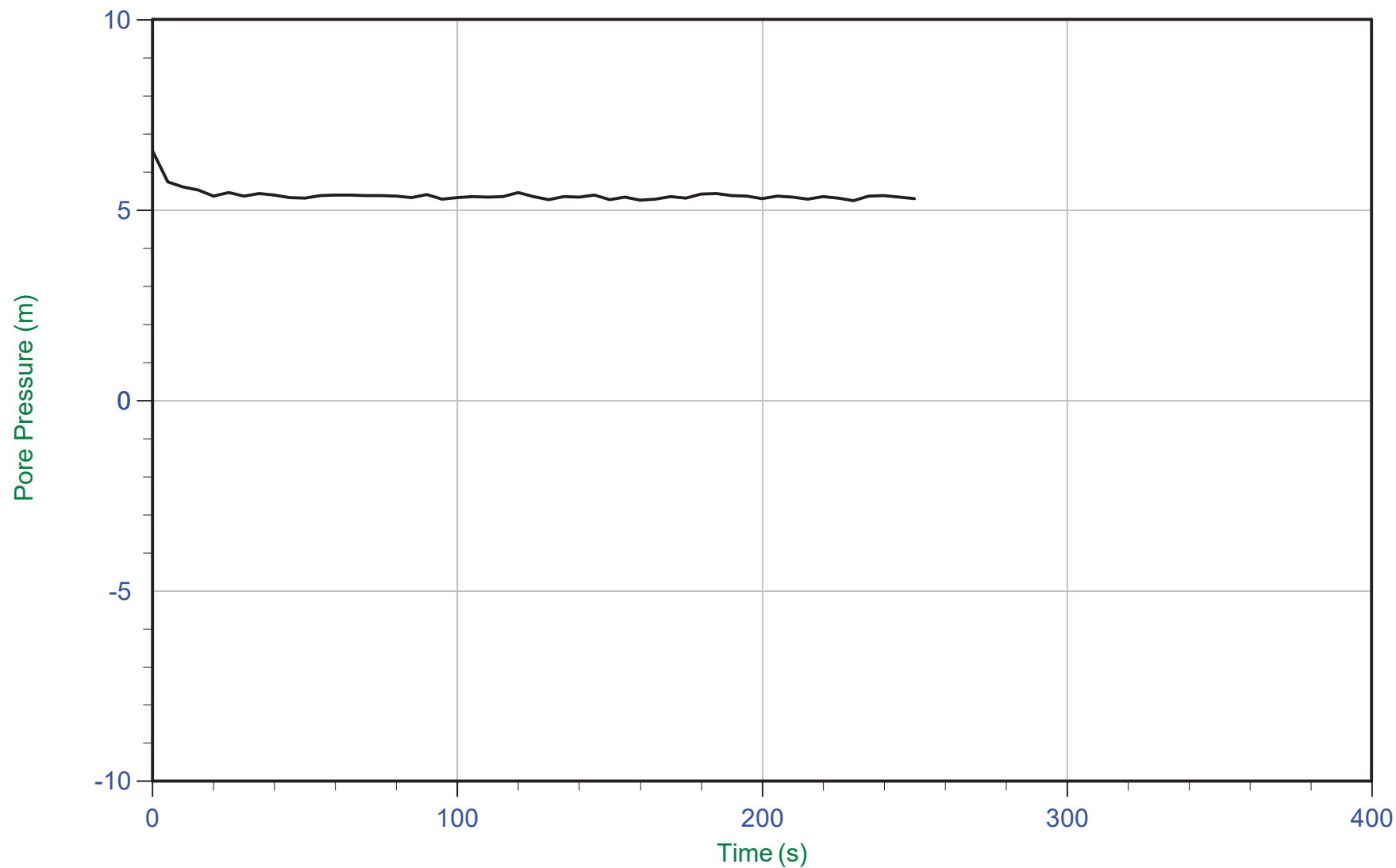
Job No: 17-05021

Date: 05/23/2017 12:56

Site: Hwy 17 - Muskrat Creek Culvert

Sounding: SCPT17-03

Cone: 379:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 17-05021_SP03.PPF

Depth: 7.000 m / 22.966 ft

Duration: 250.0 s

U Min: 5.3 m

U Max: 6.6 m

WT: 1.686 m / 5.531 ft

Ueq: 5.3 m



Thurber Engineering

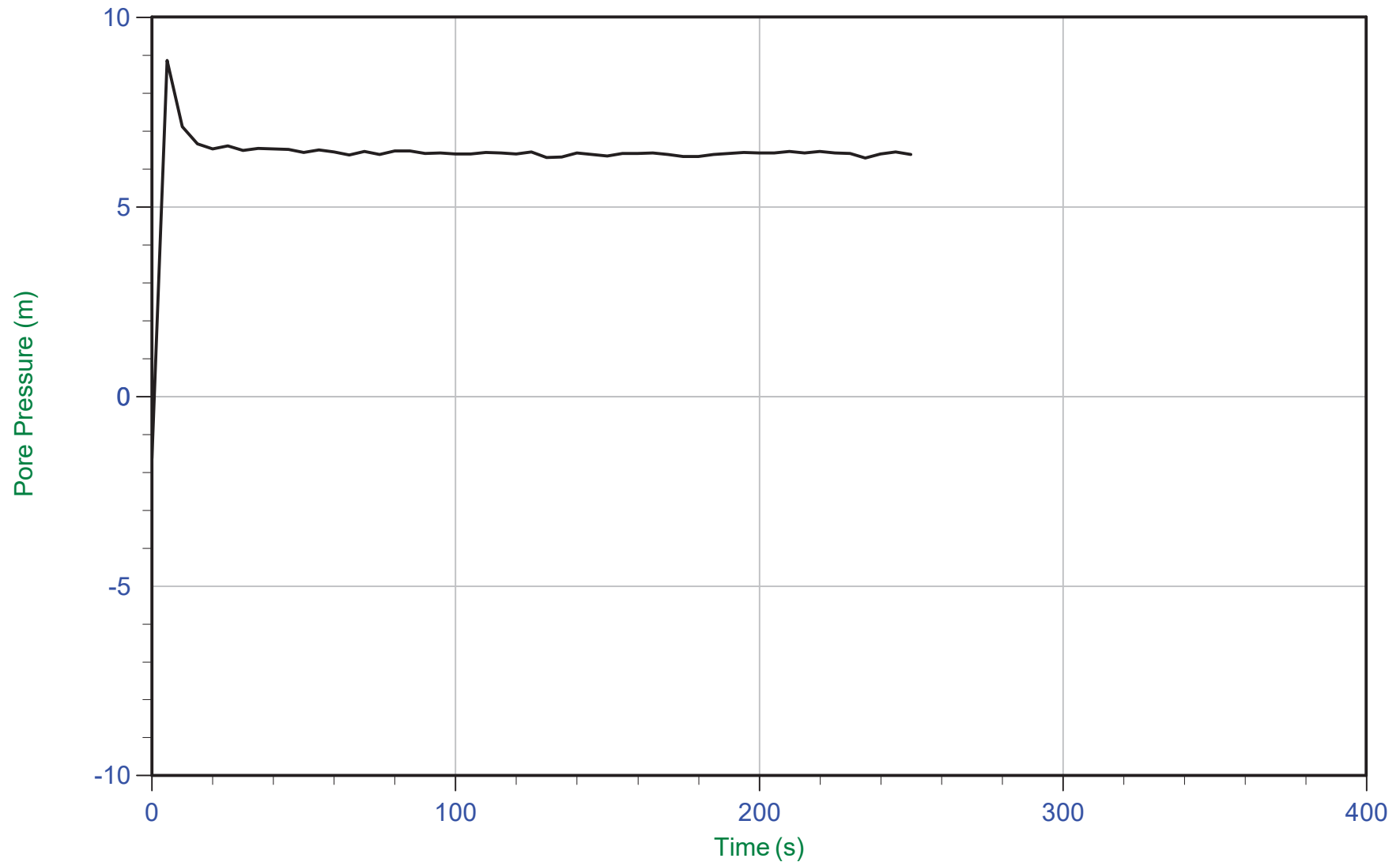
Job No: 17-05021

Date: 05/23/2017 12:56

Site: Hwy 17 - Muskrat Creek Culvert

Sounding: SCPT17-03

Cone: 379:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 17-05021_SP03.PPF
Depth: 8.000 m / 26.246 ft
Duration: 250.0 s

U Min: -1.6 m
U Max: 8.9 m

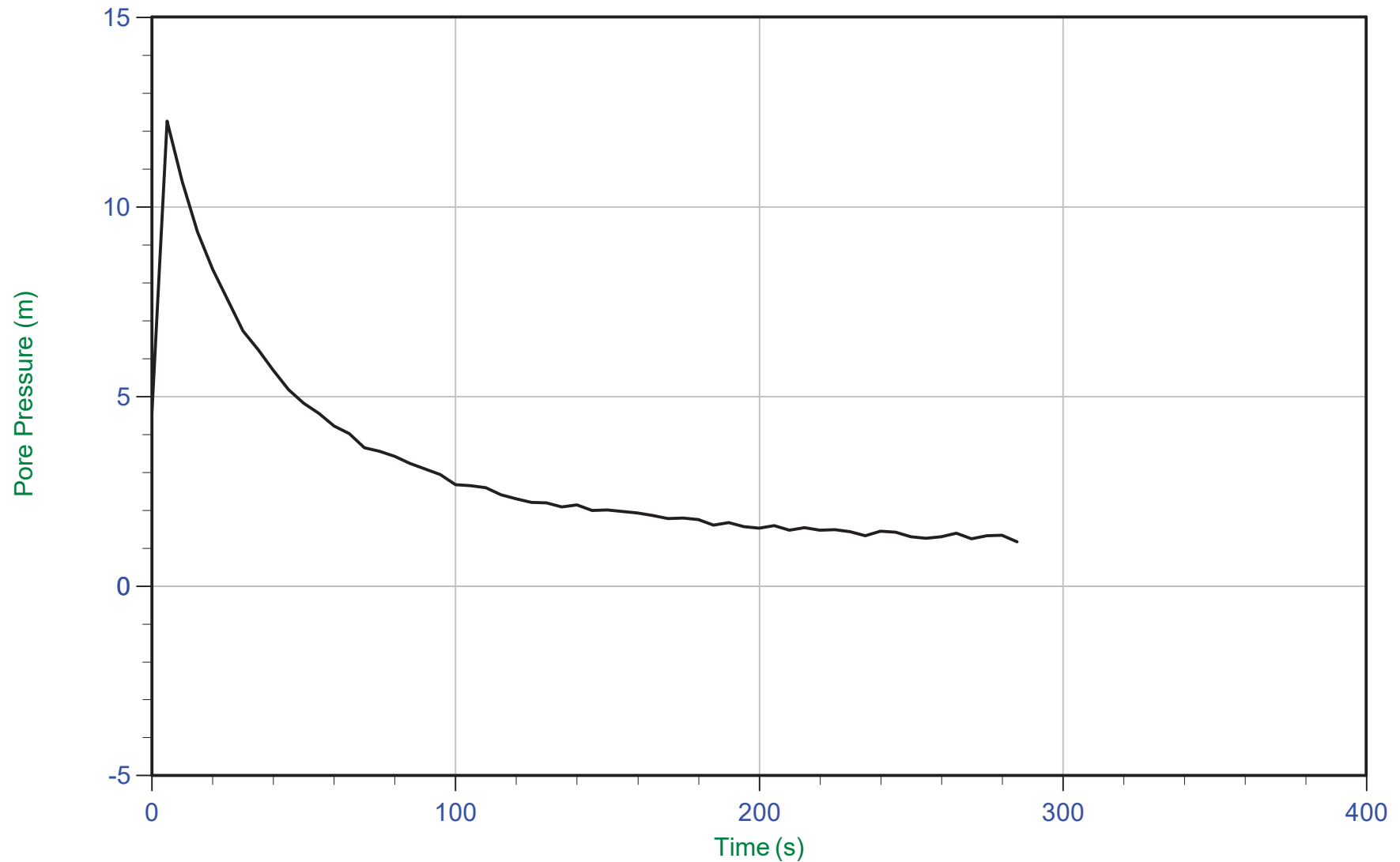
WT: 1.622 m / 5.321 ft
Ueq: 6.4 m



Thurber Engineering

Job No: 17-05021
Date: 05/23/2017 14:17
Site: Hwy 17 - Muskrat Creek Culvert

Sounding: SCPT17-04
Cone: 379:T1500F15U500 Area=15 cm²



Trace Summary: Filename: 17-05021_SP04.PPF
Depth: 2.250 m / 7.382 ft
Duration: 285.0 s
U Min: 1.2 m
U Max: 12.3 m



Thurber Engineering

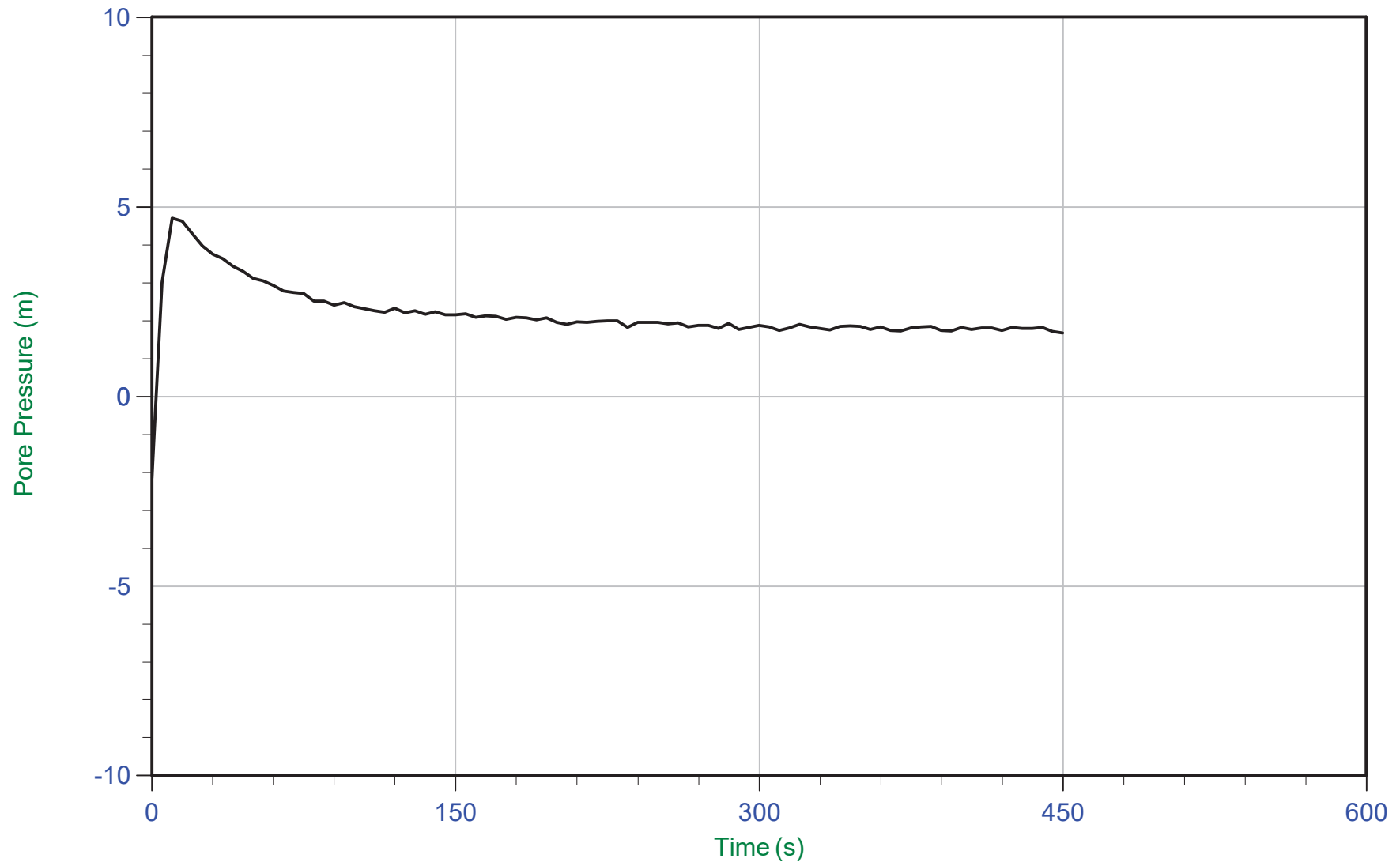
Job No: 17-05021

Date: 05/23/2017 14:17

Site: Hwy 17 - Muskrat Creek Culvert

Sounding: SCPT17-04

Cone: 379:T1500F15U500 Area=15 cm²



Trace Summary: Filename: 17-05021_SP04.PPF
Depth: 3.250 m / 10.663 ft
Duration: 450.0 s

U Min: -2.2 m
U Max: 4.7 m

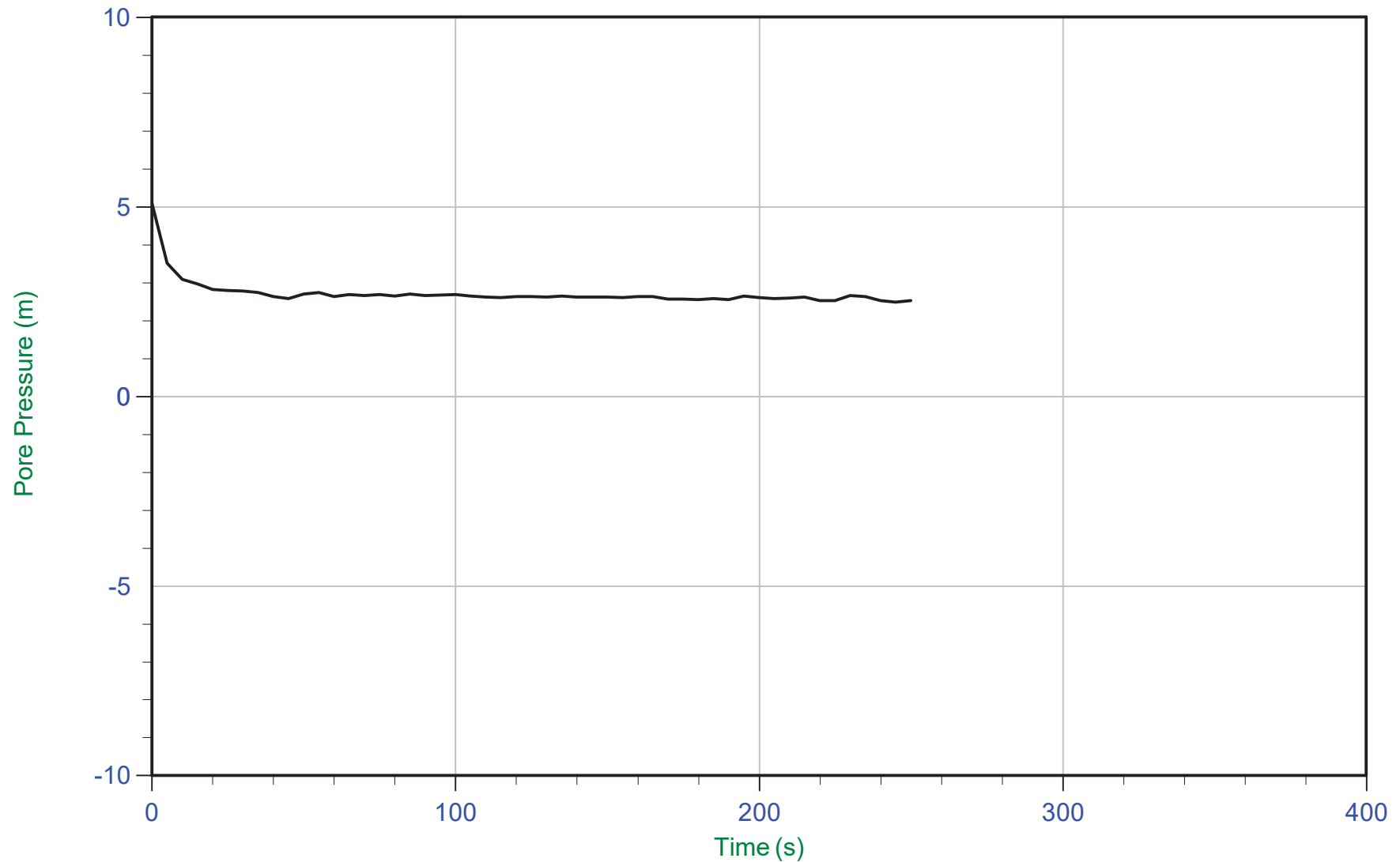
WT: 1.490 m / 4.888 ft
Ueq: 1.8 m



Thurber Engineering

Job No: 17-05021
Date: 05/23/2017 14:17
Site: Hwy 17 - Muskrat Creek Culvert

Sounding: SCPT17-04
Cone: 379:T1500F15U500 Area=15 cm²



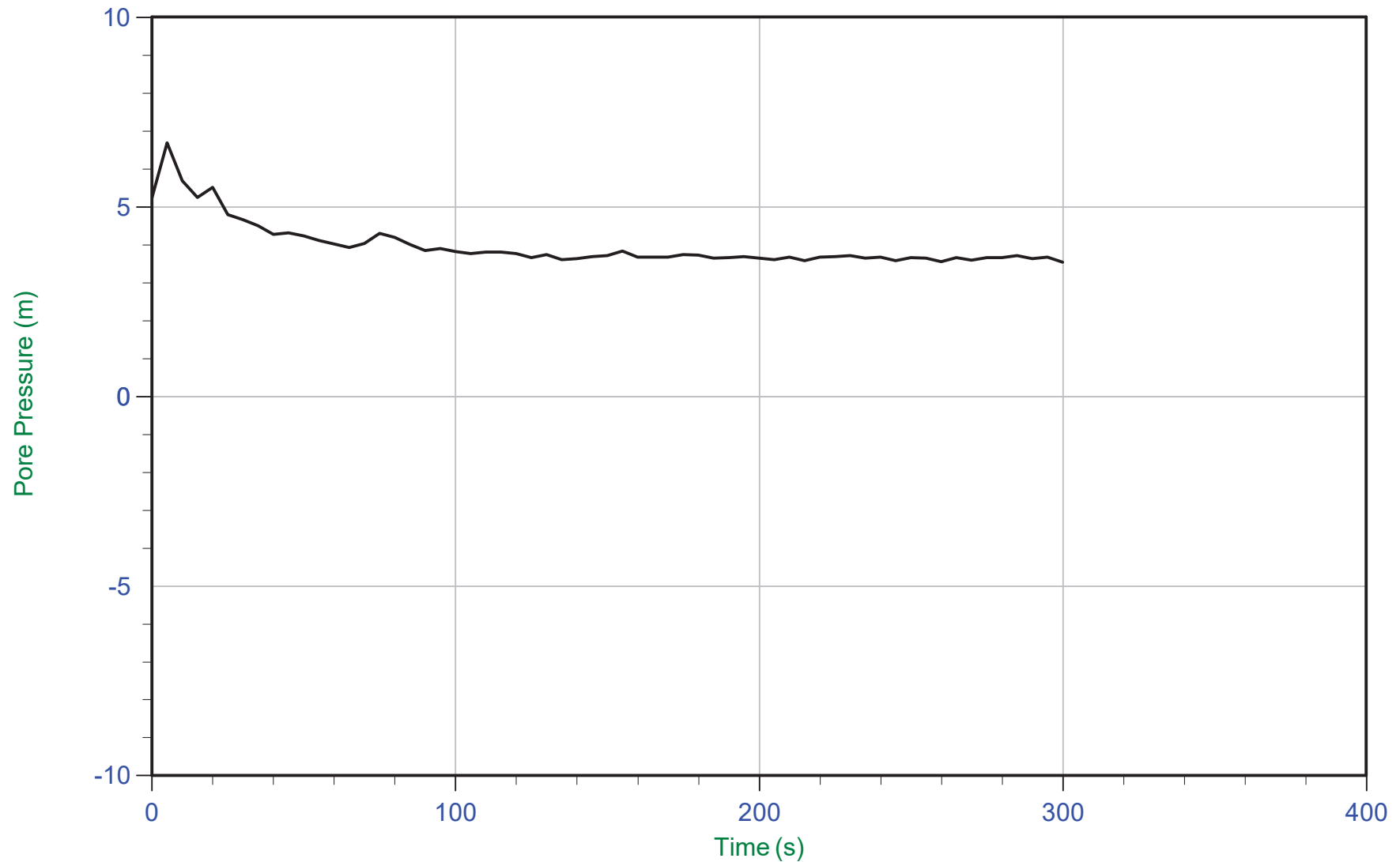
Trace Summary:	Filename: 17-05021_SP04.PPF	U Min: 2.5 m	WT: 1.675 m / 5.495 ft
	Depth: 4.250 m / 13.943 ft	U Max: 5.1 m	Ueq: 2.6 m
	Duration: 250.0 s		



Thurber Engineering

Job No: 17-05021
Date: 05/23/2017 14:17
Site: Hwy 17 - Muskrat Creek Culvert

Sounding: SCPT17-04
Cone: 379:T1500F15U500 Area=15 cm²



Trace Summary:	Filename: 17-05021_SP04.PPF	U Min: 3.5 m	WT: 1.635 m / 5.364 ft
	Depth: 5.250 m / 17.224 ft	U Max: 6.7 m	Ueq: 3.6 m
	Duration: 300.0 s		



Thurber Engineering

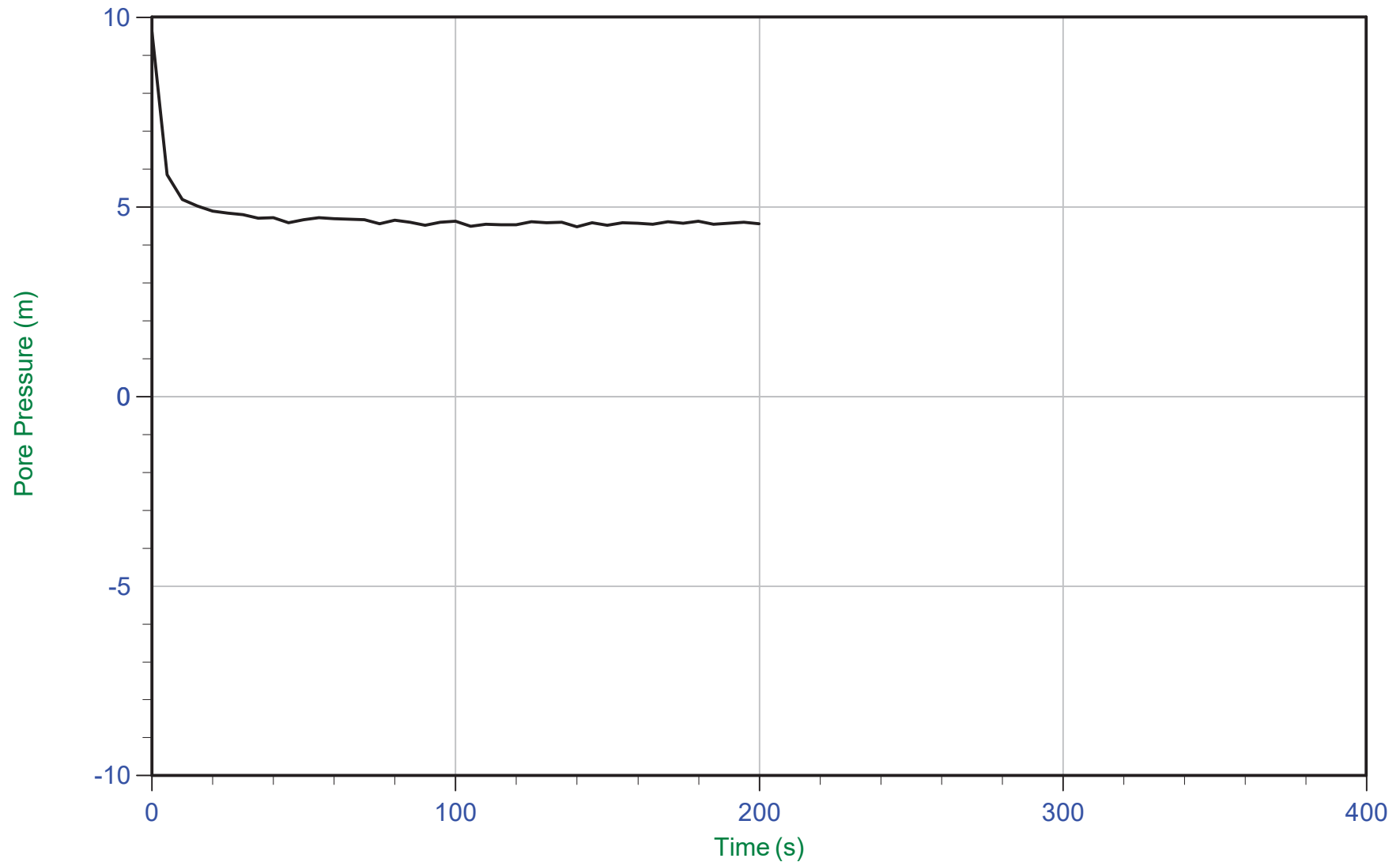
Job No: 17-05021

Date: 05/23/2017 14:17

Site: Hwy 17 - Muskrat Creek Culvert

Sounding: SCPT17-04

Cone: 379:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 17-05021_SP04.PPF
Depth: 6.250 m / 20.505 ft
Duration: 200.0 s

U Min: 4.5 m
U Max: 9.6 m

WT: 1.684 m / 5.525 ft
Ueq: 4.6 m



Thurber Engineering

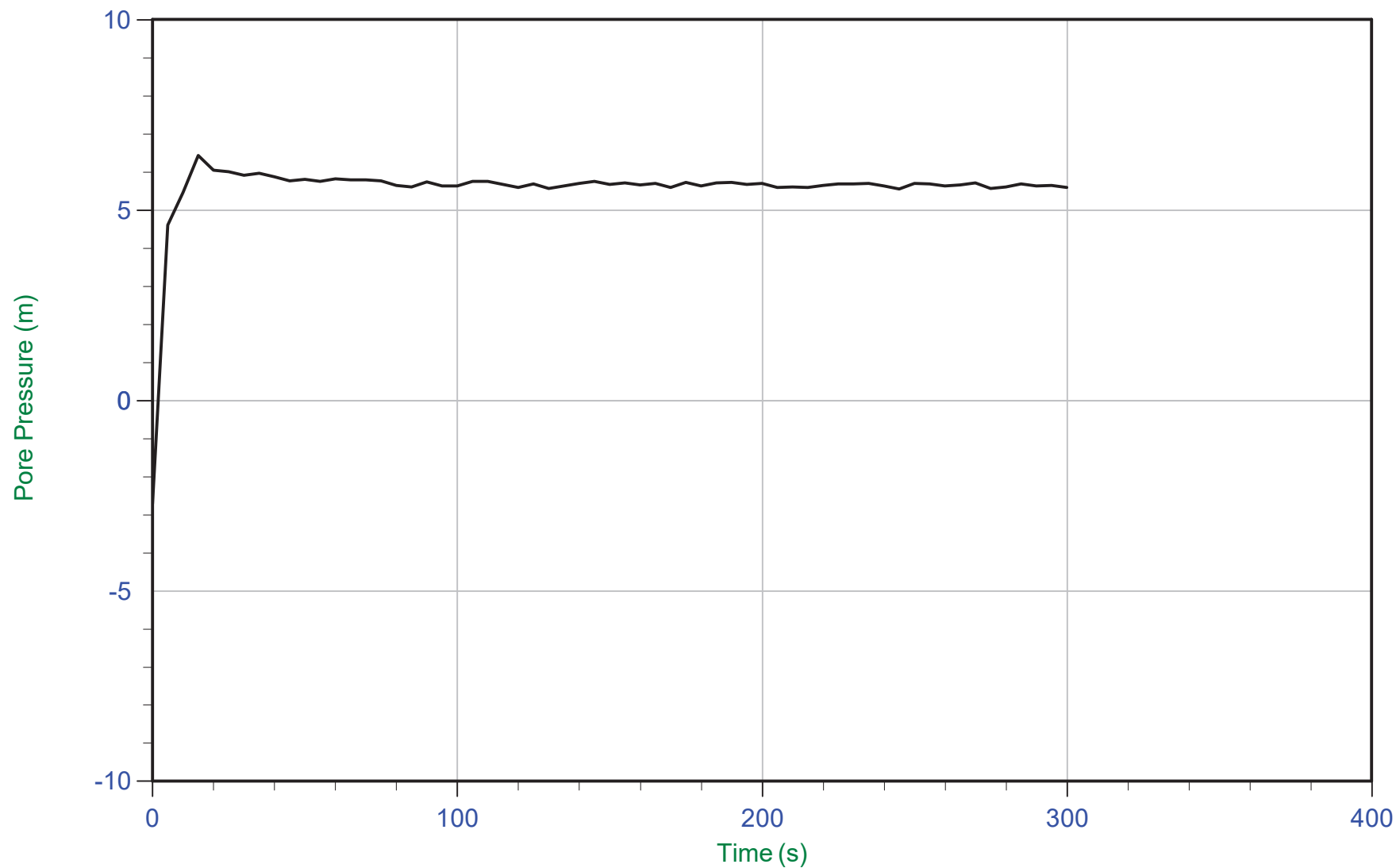
Job No: 17-05021

Date: 05/23/2017 14:17

Site: Hwy 17 - Muskrat Creek Culvert

Sounding: SCPT17-04

Cone: 379:T1500F15U500 Area=15 cm²



Trace Summary:

Filename: 17-05021_SP04.PPF

Depth: 7.300 m / 23.950 ft

Duration: 300.0 s

U Min: -2.7 m

U Max: 6.4 m

WT: 1.667 m / 5.469 ft

Ueq: 5.6 m

APPENDIX E
SITE PHOTOGRAPHS



Figure 1: Looking east along Highway 17 south embankment at the culvert outlet



Figure 2: Culvert 29-232/C outlet looking south



Figure 3: Culvert 29-232/C outlet looking northeast



Figure 4: Looking upstream at Culvert 29-232/C inlet

APPENDIX F

COMPARISON OF CULVERT REPLACEMENT ALTERNATIVES COMPARISON OF CULVERT DESIGN ALTERNATIVES

Comparison of Foundation Design Alternatives

Subexcavation to Remove and Replace Liquefiable Soils Beneath the Culvert	
Description	This alternative consists of removing the liquefiable soils from within the influence zone of the culvert foundations and replacing it with engineered fill.
Advantages	Will prevent both post-liquefaction embankment failure and post-liquefaction settlement within the influence zone of the culvert This alternative would also allow for Seismic Site Class C to be used for structural design instead of E or F Allows for the design of either an open footed or closed box culvert
Disadvantages	Post-seismic deformations away from the culvert will remain high where liquefiable soils are not removed Deep excavations required (upwards of 7.0 m) to remove liquefiable soils Sheet pile wall enclosure will be required around the full perimeter of the excavation to excavate and remove liquefiable soils Shallow bedrock encountered at the site is challenging for installation of protection system required to remove and replace the liquefiable soils Dewatering of excavation to place backfill in the dry, base instability is possible
Risks / Consequences	Unable to dewater excavation to place granular backfill / Excavation would need to be backfilled with concrete/flowable fill using Tremie techniques increasing cost and installation time
Relative Cost	Moderate
Conclusion	Not recommended for this site due to the increase in the dewatering and protection system requirements

Ground Improvement	
Description	This alternative consists of improving the existing ground conditions beneath the culvert alignment to an extent that the soil is no longer susceptible to liquefaction and provides the required bearing resistance for the culvert
Advantages	<p>Will prevent both post-liquefaction embankment failure and post-liquefaction settlement within the influence zone of the culvert</p> <p>This alternative could also allow for Seismic Site Class C to be used for structural design instead of E or F</p> <p>Allows for the design of either an open footed or closed box culvert</p> <p>Avoids the need for deep excavations beneath the base of the culvert and the costs for additional protection systems and dewatering</p>
Disadvantages	<p>Post-seismic deformations away from the culvert will remain high where ground improvement is not implemented</p> <p>Fines content of the liquefiable soil may limit effectiveness of some ground improvement options (i.e. vibroreplacement, vibrocompaction, dynamic compaction, rapid impact compaction, etc.)</p> <p>Soil mixing would likely be the only feasible ground improvement option at this site however this technique has a high cost and would be difficult to implement due the limited working platform and in-water constraints.</p> <p>Ground improvement will be expensive and will require specialist sub-contractors for a relatively small footprint</p> <p>There will be limited space to conduct a test section to validate the suitability of the proposed ground improvement method</p> <p>There will be a significant lead time and cost to design and install and verify the test section prior to production</p>
Risks / Consequences	<p>All work, including culvert construction, will need to be completed within a small fish window. / Increasing project costs and construction time</p> <p>Work do be carried out in stages and ground improvement contractor delays remobilize back to site. / Increasing project costs and construction time</p>
Relative Cost	High
Conclusion	<p>The primary advantage over the remove and replace options is that it avoids the need for deep excavations beneath the base of the culvert and the costs for additional protection systems and dewatering during construction.</p> <p>Not recommended for this site due to the relative small existing working platform and short in-water work constraints</p>

Design Entire Culvert Length to Withstand Post-Seismic Kinematic Loads	
Description	This alternative consists of leaving the liquefiable soils in place and designing the entire culvert to withstand kinematic loadings due to liquification under the design earthquake event
Advantages	Avoids the need for deep excavations beneath the base of the culvert and the costs for additional protection systems and dewatering
Disadvantages	Post-seismic deformations away from the culvert will remain high Cast-in-place culvert design is required
Risks / Consequences	All work, including culvert construction, will need to be completed within a short in-water work timeline. / The use of cast-in-place culvert design would increase construction time
Relative Cost	Moderate
Conclusion	The primary advantage of this method over remove/replace and ground improvement is that the culvert can be installed using conventional construction techniques without the need for specialized contractors Recommended

Comparison of Culvert Design Alternatives

Comment	Circular Pipes	Open Footing Culvert	Closed Box Culvert
Advantages	<p>Readily available materials and simple installation methods</p> <p>Can tolerate larger magnitude of settlement than concrete (rigid frame) culverts</p>	<p>More flexibility for installation of temporary flow passage system</p>	<p>Wide base spreads out load</p> <p>Base of the closed box does not need to be founded below frost depth reducing excavation depths</p> <p>Less prone to effects of scour and erosion</p> <p>Easier to design for post liquefaction kinematic loading</p>
Disadvantages	<p>Numerous parallel pipes required to provide hydraulic opening equivalent to existing culvert.</p> <p>Not compatible with designing to resist post-liquefaction kinematic loading.</p>	<p>Founding elevation is deeper than with closed box, requiring deeper excavation</p> <p>The founding subgrade offers relatively low bearing resistance which is insufficient based on the proposed size of the structure and height of embankment fill at this site</p>	
Risks / Consequences	<p>Potential for base disturbance if groundwater not controlled / added cost and schedule delays</p>	<p>Deeper excavation increasing excavation volume and dewatering requirements/ added cost and schedule delays</p> <p>Potential for base disturbance if groundwater not controlled / added cost and schedule delays</p>	<p>Potential for base disturbance if groundwater not controlled / added cost and schedule delays</p>
Relative Cost	Moderate	Moderate	Moderate
	NOT RECOMMENDED	NOT RECOMMENDED	RECOMMENDED

APPENDIX G

GSC SEISMIC HAZARD CALCULATION

FIGURE 1: COMPARISON OF THE CPT TEST RESULTS

FIGURE 2: COMPARISON OF CYCLIC STRESS RATIO AND CYCLIC RESISTANCE

FIGURES 3 & 4: SLOPE STABILITY ANALYSIS RESULTS

2015 National Building Code Seismic Hazard Calculation

INFORMATION: Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836
Western Canada English (250) 363-6500 Facsimile (250) 363-6565

January 17, 2018

Site: 45.5867 N, 76.8314 W User File Reference: Muskrat Creek Culvert

Requested by: , Thurber Engineering Ltd.

National Building Code ground motions: 2% probability of exceedance in 50 years (0.000404 per annum)

Sa(0.05)	Sa(0.1)	Sa(0.2)	Sa(0.3)	Sa(0.5)	Sa(1.0)	Sa(2.0)	Sa(5.0)	Sa(10.0)	PGA (g)	PGV (m/s)
0.365	0.431	0.359	0.273	0.194	0.098	0.047	0.013	0.0047	0.231	0.161

Notes. Spectral ($S_a(T)$, where T is the period in seconds) and peak ground acceleration (PGA) values are given in units of g (9.81 m/s^2). Peak ground velocity is given in m/s . Values are for "firm ground" (NBCC 2015 Site Class C, average shear wave velocity 450 m/s). NBCC2015 and CSAS6-14 values are specified in **bold** font. Three additional periods are provided - their use is discussed in the NBCC2015 Commentary. Only 2 significant figures are to be used. *These values have been interpolated from a 10-km-spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the directly calculated values.*

Ground motions for other probabilities:

Probability of exceedance per annum	0.010	0.0021	0.001
Probability of exceedance in 50 years	40%	10%	5%
Sa(0.05)	0.031	0.106	0.186
Sa(0.1)	0.045	0.138	0.231
Sa(0.2)	0.043	0.123	0.200
Sa(0.3)	0.035	0.098	0.155
Sa(0.5)	0.026	0.072	0.113
Sa(1.0)	0.013	0.038	0.059
Sa(2.0)	0.0053	0.018	0.028
Sa(5.0)	0.0011	0.0041	0.0069
Sa(10.0)	0.0006	0.0017	0.0028
PGA	0.024	0.076	0.126
PGV	0.017	0.055	0.090

References

National Building Code of Canada 2015 NRCC no. 56190;
Appendix C: Table C-3, Seismic Design Data for Selected Locations in Canada

User's Guide - NBC 2015, Structural Commentaries NRCC no. xxxxxx (in preparation)
Commentary J: Design for Seismic Effects

Geological Survey of Canada Open File 7893 Fifth Generation Seismic Hazard Model for Canada: Grid values of mean hazard to be used with the 2015 National Building Code of Canada

See the websites www.EarthquakesCanada.ca and www.nationalcodes.ca for more information

Aussi disponible en français



Natural Resources
Canada

Ressources naturelles
Canada



Figure 1: Comparison of CRR to CSR for 1:475, 1:975 and 1:2475 return period earthquake

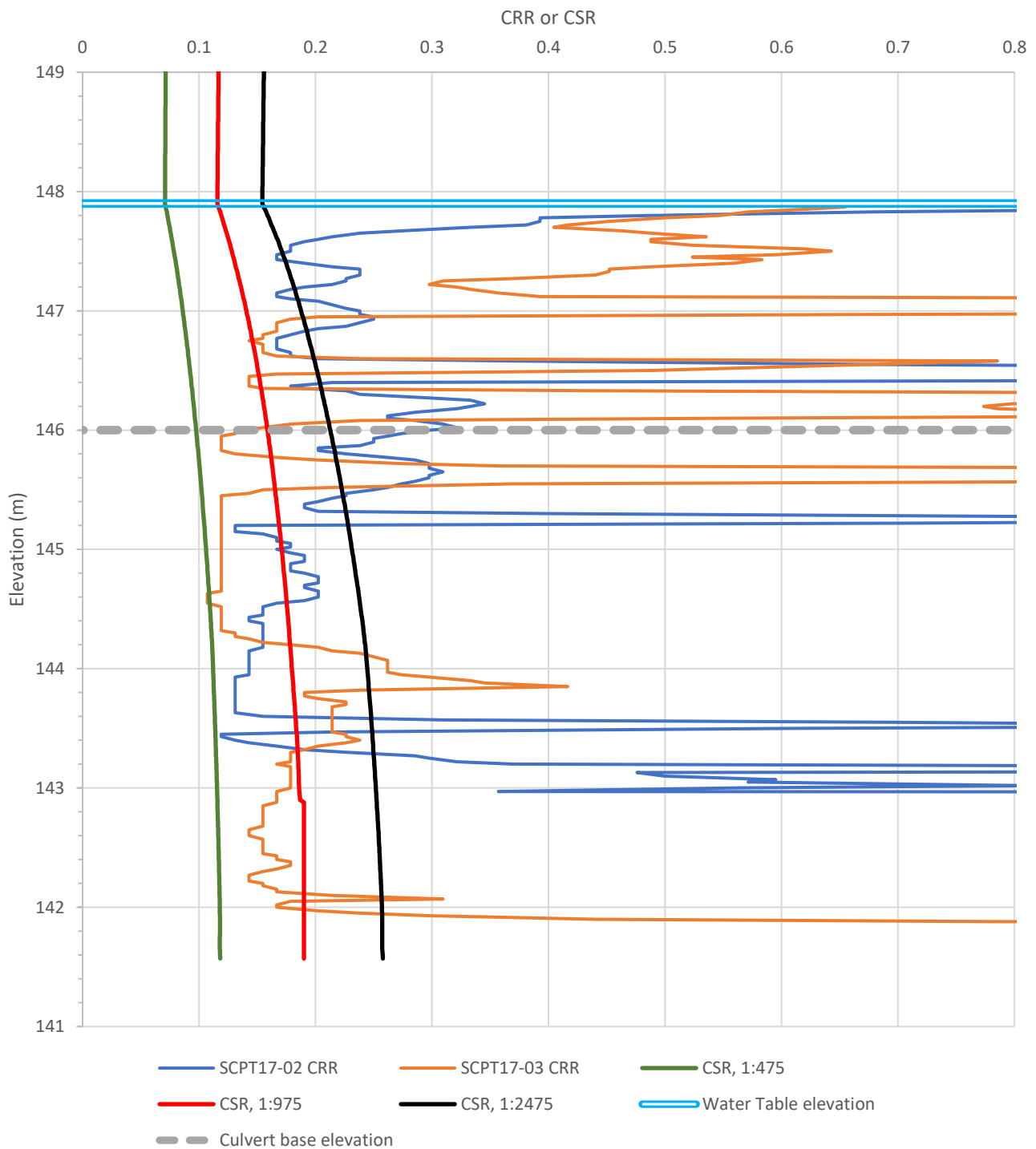
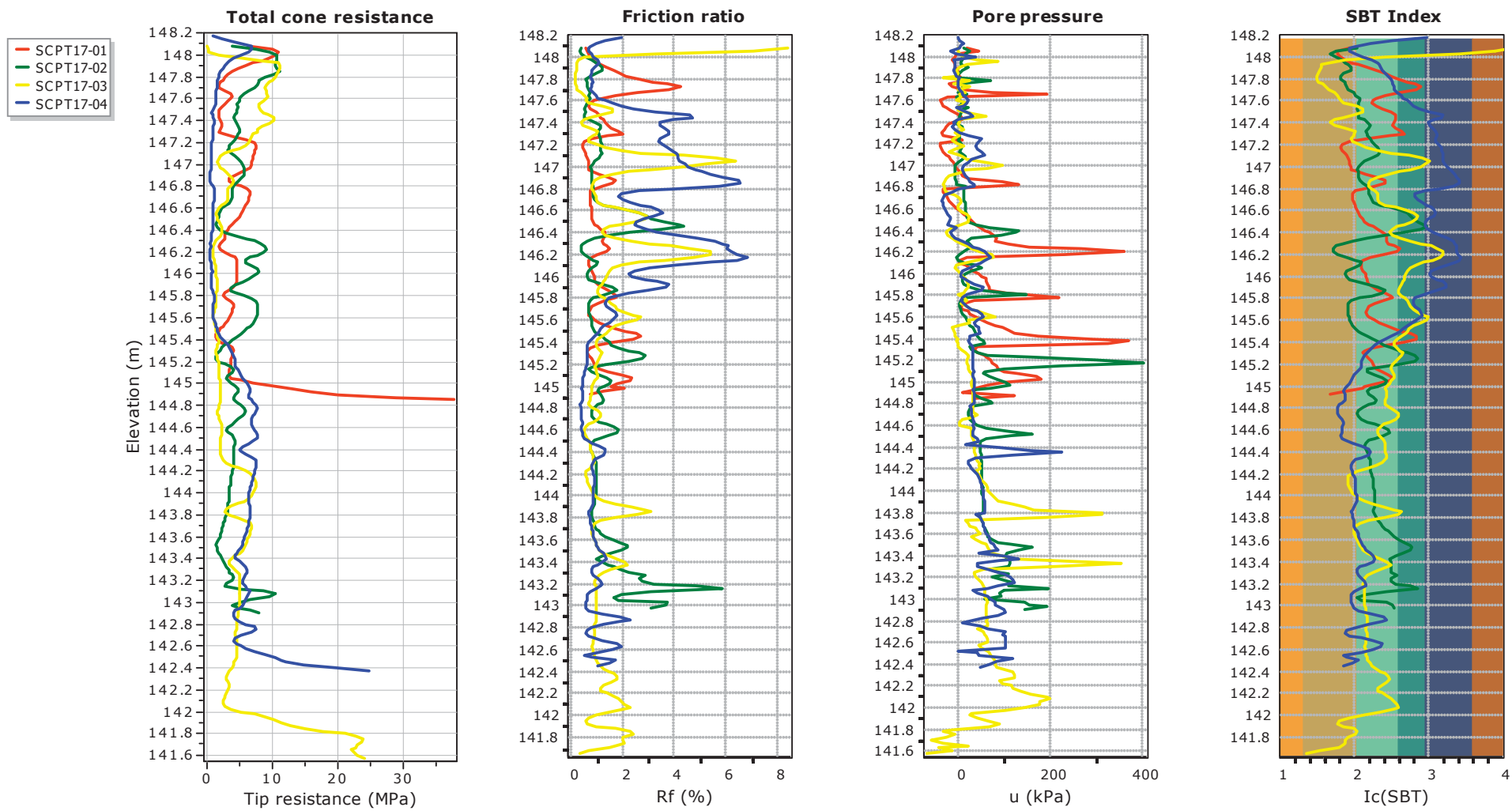


Figure 2: Overlay Basic Interpretation Plots





Project: Mega 5 - Muskrat Creek Culvert
Project No.: 19-5161-263
Task: 10

Name: South Culvert Embankment Post Liquefaction Base Case
Method: Morgenstern-Price

Created By: Simon Paxton
Checked By: DM

Name: Fill	Unit Weight: 18 kN/m ³	Cohesion: 0 kPa	Phi: 34 °
Name: Sand	Unit Weight: 18 kN/m ³	Tau/Sigma Ratio: 0.08	
Name: Silt	Unit Weight: 17 kN/m ³	Tau/Sigma Ratio: 0.08	
Name: Till	Unit Weight: 20 kN/m ³	Cohesion: 0 kPa	Phi: 36 °
Name: Bedrock			

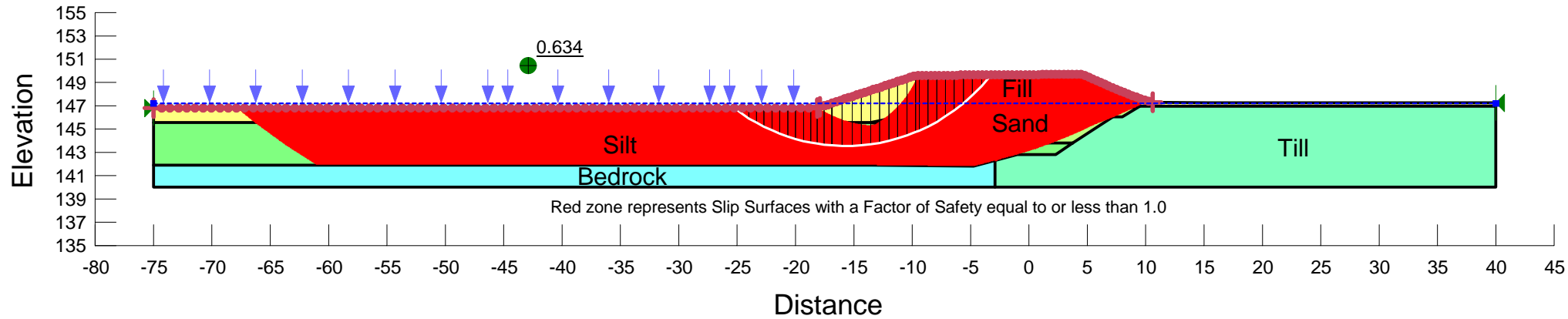


Figure 3



Project: Mega 5 - Muskrat Creek Culvert
Project No.: 19-5161-263
Task: 10

Name: South Culvert Embankment Post Liquefaction Base Case
Method: Morgenstern-Price

Created By: Simon Paxton
Checked By: DM

Name: Fill	Unit Weight: 18 kN/m ³	Cohesion: 0 kPa	Phi: 34 °
Name: Sand	Unit Weight: 18 kN/m ³	Tau/Sigma Ratio: 0.08	
Name: Silt	Unit Weight: 17 kN/m ³	Tau/Sigma Ratio: 0.08	
Name: Till	Unit Weight: 20 kN/m ³	Cohesion: 0 kPa	Phi: 36 °
Name: Bedrock			

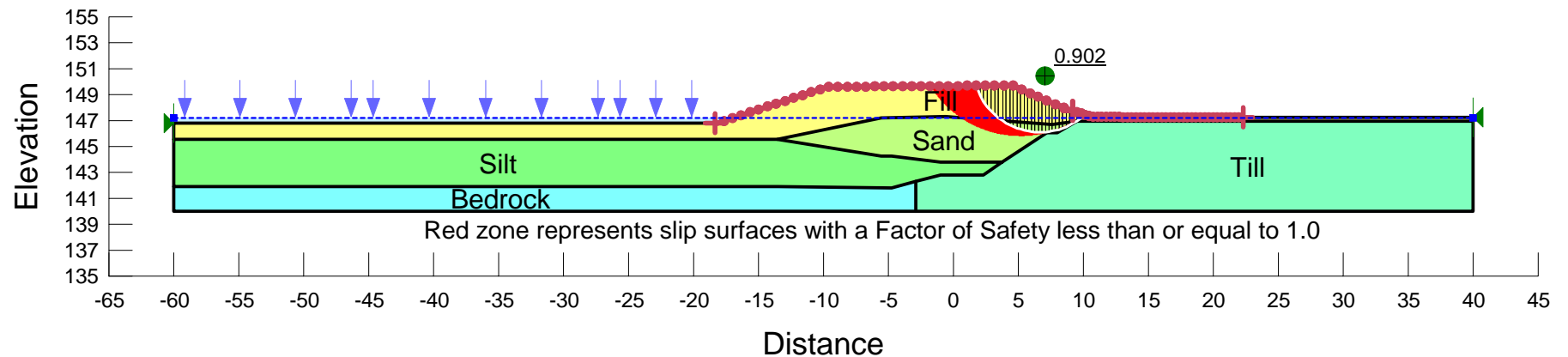


Figure 4

APPENDIX H

LIST OF REFERENCED SPECIFICATIONS NON-STANDARD SPECIAL PROVISIONS

LIST OF REFERENCED SPECIFICATIONS

OPSD 810.010	General Rip-Rap Layout for Sewer and Culvert Outlets
OPSD 3090.101	Foundation, Frost Penetration Depths for Southern Ontario
OPSS.PROV 206	Construction Specification for Grading
OPSS.PROV 404	Construction Specification for Support Systems
OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 517	Construction Specification for Dewatering
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS.PROV 804	Construction Specification for Seed and Cover
OPSS 805	Construction Specification for Temporary Erosion and Sediment Control Measures
OPSS 902	Construction Specification for Excavating and Backfilling-Structures
OPSS.PROV 1010	Material Specification for Aggregates-Base, Subbase, Select Subgrade, and Backfill Material
OPSS.PROV 1205	Material Specification for Clay Seal
OPSS 1860	Material Specification for Geotextile
Special Provision 109S12	Amendment to OPSS 902, March 2018
Special Provision 517F01	Amendment to OPSS 517, July 2017
Special Provision Foun0003	Dewatering Structure Excavations, March 2018
Design Build Special Provision 3271	Performance-Based Specification for Design and Construction of Structural Culverts

NON-STANDARD SPECIAL PROVISIONS

RECOMMENDED WORDING FOR “NSSP – A CONCRETE WORKING SLAB”

This Non-standard Special Provision covers the requirements for the supply and placement of a concrete working slab to protect the sand and silt subgrade of the entry/exits pits and provide a proper working surface for the tunnelling equipment.

Excavation for the working slab shall be according to OPSS.PROV 902. Within four hours following inspection and approval of the prepared subgrade, a working slab with a minimum thickness of 100 mm shall be placed on the foundation subgrade as specified in the Contract Documents. Concrete for working slabs shall have a minimum 28 day strength of 20 MPa.

RECOMMENDED WORDING FOR “NSSP – 902.07.05 Excavation”

Subsection 902.07.05 of OPSS 902 is amended by the addition of the following:

Excavations at the site may encounter bedrock and bedrock excavation should be anticipated. It should be noted that the bedrock profile varies considerably across the site from the south to north. The Contractor shall be prepared to excavate and remove the bedrock at the site to extend the excavations to the design depths.

Reference can be made to the Foundation Investigation Report for the Replacement of Structural Culvert No. 29-232/C Muskrat Creek Crossing of Highway 17, prepared by Thurber Engineering Ltd., 2018, for further details on likely subsurface conditions at the foundation locations.

RECOMMENDED WORDING FOR “NSSP – TEMPORARY PROTECTION SYSTEM”

It should be noted that the bedrock profile varies considerably across the site. Temporary protection systems will be installed in ground conditions that include sloping bedrock. The Contractor's installation method and temporary protection system design must take into account the existing soil and bedrock conditions.

Reference can be made to the Foundation Investigation Report for the Replacement of Structural Culvert No. 29-232/C Muskrat Creek Crossing of Highway 17, prepared by Thurber Engineering Ltd., 2018, for further details on likely subsurface conditions at the foundation locations.